PB-223 726

# EVALUATION OF TREATMENT FOR URBAN WASTEWATER REUSE

K. D. Linstedt, et al

Colorado University

Prepared for:

Environmental Protection Agency

July 1973

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PB 223 726

EPA-R2-73-122
July 1973

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# Evaluation Of Treatment For Urban Wastewater Reuse

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EPA-R2-73-122
July 1973

# EVALUATION OF TREATMENT FOR URBAN WASTEWATER REUSE

by

K. D. Linstedt
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University of Colorado
Boulder, Colorado 80302

Grant #17080 DOI

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for the

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# ABSTRACT

This study was undertaken to evaluate the efficacy of combining biological nitrification with each of two modes of chemical clarification for production of a water suitable for specific industrial reuse applications.

For this purpose, a 7200 gpd pilot plant was constructed, operated and analyzed. The nitrifying activated sludge system was operated to identify the effect of variations in detention time, loading, and temperature on the performance of the system. In the temperature range of 5-30°C, a maximum nitrification rate was observed to occur at about 25°C. Variations in BOD, COD, and ammonia loadings were investigated over a wide range, with complete nitrification observed for loadings at or below the following levels: 0.4 $\#BOD_5/\#MLSS/DAY$; 1.4 #COD/#MLSS/DAY; and 0.16 $\#NH_3-N/\#MLSS/DAY$. The oxygen demand for ammonia oxidation by nitrification was determined to be 4.6 $\#O_2/\#NH_3-N$.

In the conventional clarification system, low doses of lime or alum were shown to be effective in removing greater than 95% of the BOD, turbidity, and suspended solids from the nitrified secondary effluent. Efficient phosphorus removals necessitated higher lime and alum additions of 300 mg/l, and 100-150 mg/l, respectively. At the higher lime doses, a two-fold reduction in bacterial organisms was achieved. Specific heavy metals were removed through both alum and lime additions. With either coagulant chemical, practical rapid sand filter runs of 16 hours were observed to be possible.

Similar removal results were obtained with alum in a high-rate clarification flow system with direct dual-media filtration of alum flocculated wastewater. Filter runs of 5-6 hours could be realized.

This report was submitted in fulfillment of Project Number 17080 - DOI, under sponsorship of the Office of Research and Monitoring, Environmental Protection Agency, by the University of Colorado, Boulder, Colorado.

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**SELECTED WATER RESOURCES ABSTRACTS**
**INPUT TRANSACTION FORM**

1. Report No.
2.
3. Accession No.
W 73-14750

4. Title
EVALUATION OF TREATMENT FOR URBAN WASTEWATER REUSE,

5. Report Date
6.
8. Performing Organization Report No.

7. Author(s)
Linstedt, K. D., and Bennett, E. R.

10. Project No.
EPA Project 17080 DOI

9.
Colorado University, Boulder, Dept of Civil and Environmental Engineering

11. Contract/Grant No.
~~Grant 17080 DOI~~

13. Type of Report and Period Covered

12. Sponsoring Organization

15. Supplementary Notes
Environmental Protection Agency, ~~Office of Research and Monitoring~~ Environmental Protection Technology Series, Report EPA-R2-73-122, July 1973. 139 p, 73 fig, 8 tab, 51 ref.

16. Abstract
The efficacy of combining biological nitrification with each of two modes of chemical clarification is evaluated for production of a water suitable for specific industrial reuse applications. A 7200 gpd pilot plant was constructed, operated and analyzed. In the conventional clarification system, low doses of lime or alum were effective in removing greater than 95% of the BOD, turbidity, and suspended solids from the nitrified secondary effluent. Efficient phosphorus removals necessitated higher lime and alum additions of 300 mg/liter, and 100-150 mg/liter, respectively. At the higher lime doses, a two-fold reduction in bacterial organisms was achieved. Specific heavy metals were removed through both alum and lime additions. With either coagulant chemical, practical rapid sand filter runs of 16 hours were ~~observed to be~~ possible. Similar removal results were obtained with alum in a high-rate clarification flow system with direct dual-media filtration of alum flocculated wastewater. Filter runs of 5-6 hours could be realized. (Woodard-USGS)

17a. Descriptors
*Sewage treatment, *Waste water treatment, *Water reuse, Model studies, Methodology, Water conservation, Water pollution control, Water utilization, Recirculated water, Industrial water, Reclaimed water, Treatment facilities

17b. Identifiers

17c. COWRR Field & Group
05D

18. Availability
also from
GPO, Washington, D C
20402 Price $2.10 microfiche from NTIS
PB223 726 $1.45
EP1.23/2: 73-122

19. Security Class
21. No. of Pages
22. Price

Send To:
**WATER RESOURCES SCIENTIFIC INFORMATION CENTER**
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**WASHINGTON, D.C. 20240**

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WRSIC 102 (REV. JUNE 1971)

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# I. CONCLUSIONS

It was demonstrated in this pilot plant investigation that complete conversion of ammonia to nitrate can be achieved through biochemical oxidation in a second-stage activated sludge type system.

The nitrification reaction rate is quite predictable with a reaction rate constant of 0.16 $\#NH_3/\#MLSS/DAY$ and aeration requirements of 4.6 $\#O_2/\#NH_3$-N. It appears that the rate of reaction is not proportional to the ammonia concentration, but proceeds at a constant rate until all of the $NH_4$ is converted to $NO_3^-$. With the relatively rapid rate of ammonia conversion, and a high organism level it should be possible to remove the second-stage BOD from secondary effluents in a comparatively small tertiary activated sludge unit.

The rate of nitrification is temperature dependent, with a maximum rate at about 25°C.

The settleability of the nitrifying sludge improves with increasing temperature in the range of 5-30°C.

Acclimation of nitrifying activated sludge is considerably more difficult than in a carbonaceous system. In this study, the acclimation time was reduced somewhat by seeding the aeration system with aerobically digested activated sludge.

In the tertiary nitrification system, the sludge has a very high biomass fraction. This sludge is relatively light, resulting in a pinpoint floc carry-over problem. With the solids loss resulting from this carry-over, coupled to the long generation time of the nitrifying organisms, it was not possible in this study to achieve a net sludge growth. Therefore, in plant applications of nitrifying activated sludge systems it may be necessary to provide for enhancement of the flocculation in the biological system, or to maintain a supplemental source of nitrifying organisms.

In the chemical clarification studies utilizing sedimentation-filtration treatment it was shown that moderate $Ca(OH)_2$ additions of 100 mg/ℓ, with corresponding pH levels of 9.0-9.5, produce a clear effluent with a high degree of suspended solids and BOD removal. Higher lime doses of 200-300 mg/ℓ (pH>11.0) are necessary to reduce the total phosphate level below 1 mg/ℓ. At these higher lime doses the residual COD can be reduced to about 20 mg/ℓ as a limit.

The effectiveness of lime in reducing the concentration of bacterial organisms increases with dose and pH. At lime doses of 400 mg/ℓ, fecal coliform, fecal streptococcus, and total coliform concentrations can be reduced by two orders of magnitude utilizing conventional clarification without chlorination.

Lime is effective in precipitating some heavy metals from secondary effluent. The concentrations of iron, manganese, chromium, and zinc are reduced in lime treatment, while copper, lead, molybdenum, aluminum, and cadmium are not significantly precipitated at the trace concentrations found in wastewater.

At typical suspended solids levels of 5 mg/ℓ discharging from a chemical clarifier, rapid sand filter runs of 16 hours can be expected in tertiary treatment clarification at loading rates of 2 gpm/ft$^2$.

Lime clarification of the nitrified secondary effluent investigated in these studies resulted in the production of waste sludge at the rate of 0.56 tons/M.G./100 mg/ℓ $Ca(OH)_2$. This correspo closely to the stoichiometric quantity of $CaCO_3$ associated with the applied lime dose.

In both the conventional and high-rate clarification systems alum was effective in producing a water with good clarity and low residual BOD levels at an alum dose of 50 mg/ℓ. To achieve the maximum phosphate removals an alum dose of 100-150 mg/ℓ was required. This corresponds to an $Al/PO_4$ ratio of about 1.5.

The maximum alum effectiveness occurs in the pH range of 5-6.

Wastewater concentrations of iron, manganese, and chromium are significantly reduced by alum while lead, zinc, cadmium, molybdenum, and copper concentrations are not appreciably altered. The concentration of aluminum increases.

Bacterial removals of about 90% can be achieved through alum clarification, irrespective of the alum dose.

With the nitrified secondary effluent investigated in this study, practical dual-media filter runs of 5-6 hours can be achieved for loading rates up to 4 gpm/ft$^2$. At higher loading rates, the headloss develops much faster in the alum-polymer system because the alum floc breaks apart and carries through to the sand surface.

Filter runs with alum applied in the conventional clarification mode are comparable to those indicated for lime treatment in that system.

It was demonstrated that both the conventional and high-rate clarification systems are capable of developing a clear product water which would be acceptable for use in several industrial applications.

## II. RECOMMENDATIONS

From the results obtained in this and other research, it is evident that chemical clarification with or without filtration holds considerable promise for upgrading existing wastewater effluents at a comparatively low cost. With addition of moderate doses of alum or lime, a clear product can readily be produced having a $BOD_5$ of less than 5 mg/$\ell$. This system also has capability for phosphorus removal through the simple manipulation of the applied chemical dose. The choice between alum and lime should be considered in the context of each specific operational and sludge disposal situation, since these criteria will likely dictate the economic practicality of the system. The use of this technology should be strongly encouraged for improved water pollution control and increased opportunities for beneficial successive use of products from such plants.

Further, for potential high-level successive use applications, including potable use, the studies elaborated in this and other reports should be extended to evaluate the role of the nitrification and chemical clarification systems as pre-treatment to subsequent carbon and ion removal units. This and alternate process trains should be evaluated in the context of successive potable use as well as for their water pollution control potential, since there appears to be increasing pressure developing for general application of the reuse concept. Necessarily, the evaluation for potable reuse potential must be much more extensive than that required by water pollution control criteria.

## III. INTRODUCTION

Water pollution and its control are of great concern with the increasing quantities and continually changing character of water pollutants. Suspended solids, refractory organics, nutrients, and inorganic salts are discharged to receiving streams in ever increasing quantities. Simultaneously, the streams are being overwhelmed by the higher load of biologically degradeable pollutants. The 85 to 90 percent BOD removal attainable by conventional waste treatment practices is becoming inadequate for maintenance of aesthetically acceptable conditions in many receiving streams.

Associated with the indicated degradation of water quality is a reduction of suitable supply water, since the usefulness of a stream for supply purposes is closely related to the stream quality. Renovation and successive use of wastewater offer one means of alleviating both the water pollution and water supply problems.

Successive use is particularly attractive in many of the arid western cities, where legal constraints and water development costs make new sources of raw water increasingly difficult to acquire. For cities on the eastern slope of the Rocky Mountains, source water is presently transported through the Continental Divide. Collection systems extend for more than 100 miles and involve complex networks of tunnels, canals, conduits, and reservoirs. Thus, this area has many of the characteristics of a good location for initiation of large scale fresh water - renovated water tradeoffs for satisfying industrial needs.

In this context, the objective of this study has been to evaluate the technical feasibility of several treatment processes for upgrading wastewater quality to the point that it is available for certain industrial reuse applications. Specific processes included in this initial phase of evaluation were biological nitrification, and chemical clarification utilizing either conventional or high-rate filtration. For these treatment systems, data have been collected to assess the practicality of upgrading secondary effluent at the Metropolitan Denver Wastewater Treatment Plant to a quality level which is acceptable in selected industrial water applications. Specifically, the aim was to: 1) define the reasonable ranges of these parameters which control the biological nitrification activated sludge process in order to identify the practical limits of the important operating variables; and, 2) to determine the optimum dose, pH, mixing, and filtration conditions for achieving suspended solids and phosphate removals in both the conventional

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and high-rate clarification systems. In the fulfillment of these goals, a 7200 gpd pilot plant was developed, operated, and analyzed.

## IV. EXPERIMENTAL PILOT PLANT

### INTRODUCTION

The 5 gpm pilot plant utilized in these studies was constructed on the site of the Metropolitan Denver Sewage Disposal District Treatment Plant. The units included in this plant are shown in the simplified flow schematic of Figure 1. As indicated in the Figure, the 5 gpm influent flow consisted of secondary effluent from a municipal activated sludge plant. This influent was initially subjected to biological nitrification for oxidation of the ammonia nitrogen form to nitrate ion. Following nitrification, the flow was divided into two streams for subsequent clarification treatment. One of the systems had provision for several operations which are similar to conventional water treatment operations. These included chemical addition, rapid mix, flocculation, clarification, and filtration. A parallel treatment sequence was designed as a high-rate water clarification system. This treatment system incorporated chemical addition, rapid mix, flocculation, polyelectrolyte addition, and dual-media filtration, following the initial biological nitrification process.

Flow in the plant was gravity from the nitrification unit to the filter inflow. Pumps were provided to overcome the hydraulic head-loss on the filters. This hydraulic profile for the system is indicated in Figure 2.

### PILOT PLANT DESCRIPTION

The pilot plant influent was obtained from the Parshall flume preceding the chlorination basin at the Metro Denver Plant. The waste was pumped with a Teel 1P555 screw type pump driven with a variable speed pulley by a ½ H.P. electric motor. The plant influent flow was transported through 3/4" PVC pipe to the Denver Water Department effluent pumping station in which the pilot plant was located. The line entered the building where the pipe enlarged to 1¼". At this point there was a ¼" globe valve sample cock for sample removal. In subsequent discussions of the system this point is referenced as sample point No. 1. Subsequent to this point the system piping was 2" schedule 80 PVC unless otherwise noted. Following the sample cock the flow passed through a 2" PVC globe valve and into the nitrification tank.

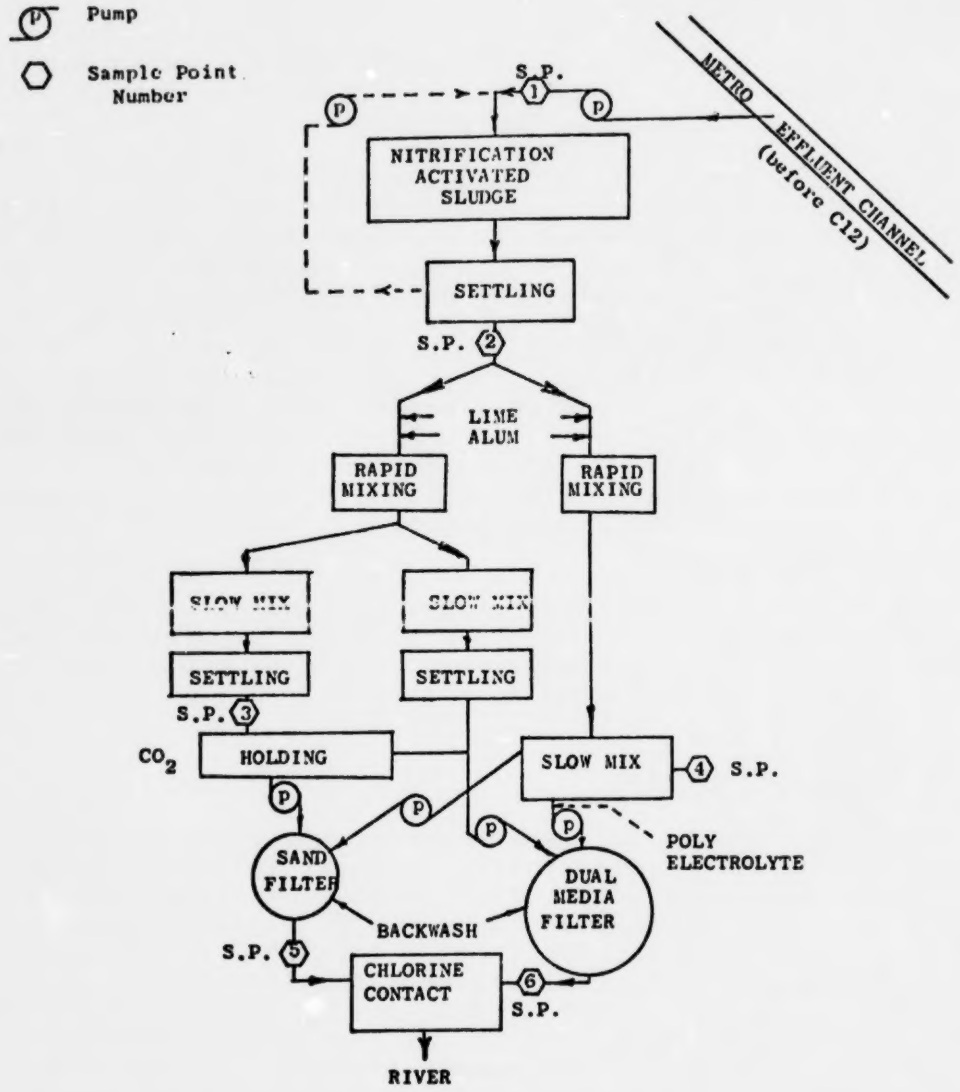
P Pump
Sample Point Number
S.P. 1
P
P
METRO EFFLUENT CHANNEL (before C12)
NITRIFICATION ACTIVATED SLUDGE
SETTLING
S.P. 2
LIME
ALUM
RAPID MIXING
RAPID MIXING
SLOW MIX
SLOW MIX
SETTLING
SETTLING
S.P. 3
CO2
HOLDING
SLOW MIX
4 S.P.
P
P
P
P
POLY ELECTROLYTE
SAND FILTER
DUAL MEDIA FILTER
BACKWASH
S.P. 5
CHLORINE CONTACT
6 S.P.
RIVER

Figure 1

PILOT PLANT FLOW SCHEMATIC

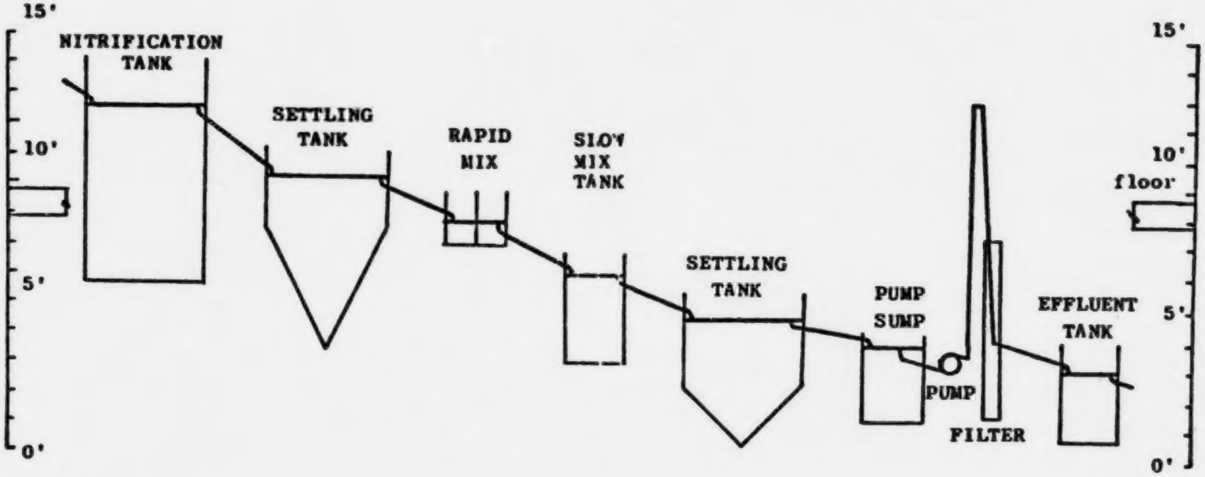
15'
10'
5'
0'
NITRIFICATION TANK
SETTLING TANK
RAPID MIX
SLOW MIX TANK
SETTLING TANK
PUMP SUMP
PUMP
FILTER
EFFLUENT TANK
floor
15'
10'
5'
0'

Figure 2

HYDRAULIC PROFILE

DENVER REUSE PILOT PLANT STUDY

As indicated, the first unit operation in the Pilot Plant series was a biological nitrification system. It consisted of a typical activated sludge system including an aeration tank followed by a clarifier. Two different aeration systems were used in different parts of the study. The primary system consisted of a three compartment unit having a total volume of 145 $ft^3$. It is shown in Figure 3 with the nitrification clarifier. At the design flow rate of 5 gpm, these system components had detention times of 3 hours, and 70 minutes, respectively. The second aeration system consisted of a series of six tanks of 50 gallon capacities.

The primary nitrification aeration tank was constructed of 3/16" steel plate 4'6" square by 7'7" deep. The tank was baffled into 3 sections by steel plates extending from top to bottom. Flow entered at the bottom of the tank and the first baffle had an opening at the top just below the minimum water level to provide for the discharge from the first pass. Likewise, the second baffle had an opening at the bottom, with the flow directed up and over the overflow weir at the end of the third pass. The outlet weir was a V-notch weir which extended along one side of the tank and was 12" deep by 2" wide. The weir height was adjustable over a range of approximately ±20% of the design detention time of the tank. The weir assembly is shown in Figure 4. From the overflow weir a 2" wire reinforced flexible hose directed the process flow to a fitting through the side of the tank. The mixed liquor in the nitrification tank was aerated by six Chicago Pump Precision saran wrapped tube diffussors supplied by the Metro Denver Sewage Treatment Plant. Air was supplied at 16 cfm by a Roots Moxair 1702-162 rotary compressor driven by a 2 H.P. motor. In this aeration unit, effluent from the Metro Denver Plant was subjected to aeration with a bacterial activated sludge having a predominance of Nitrosomonas and Nitrobacter.

Following the three hour aeration period, the suspended biological solids passed into a clarifier where the biological solids were removed from the liquid stream and the nitrified effluent was discharged through an overflow weir. The settled bacterial solids were continuously recycled into the influent of the aeration unit to provide a constant nitrifying bacterial population for steady-state bacterial nitrification. From the aeration tank the process flow passed into the biological clarifier. A 2" PVC globe valve and a ¼" sample cock were provided between the aeration tank and the clarifier.

It was found necessary to provide piping within the clarifier to impart a radial component to the flow in order to

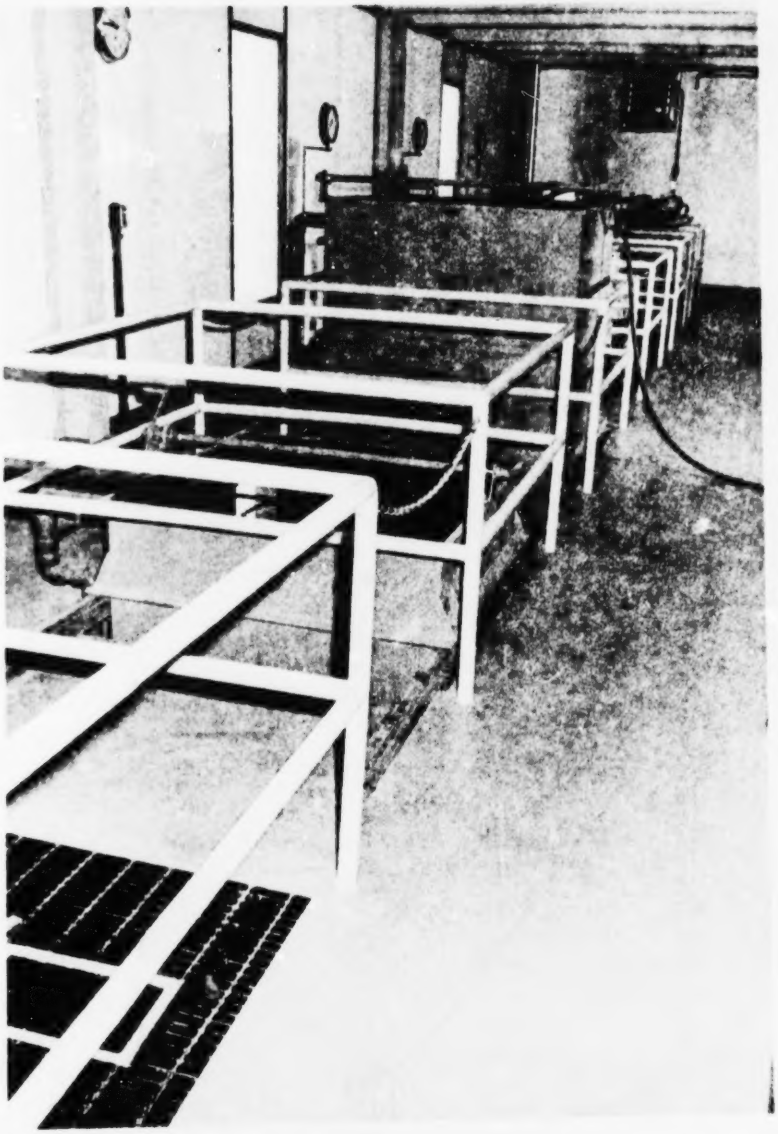

Figure 3

NITRIFICATION AERATION AND CLARIFICATION UNITS

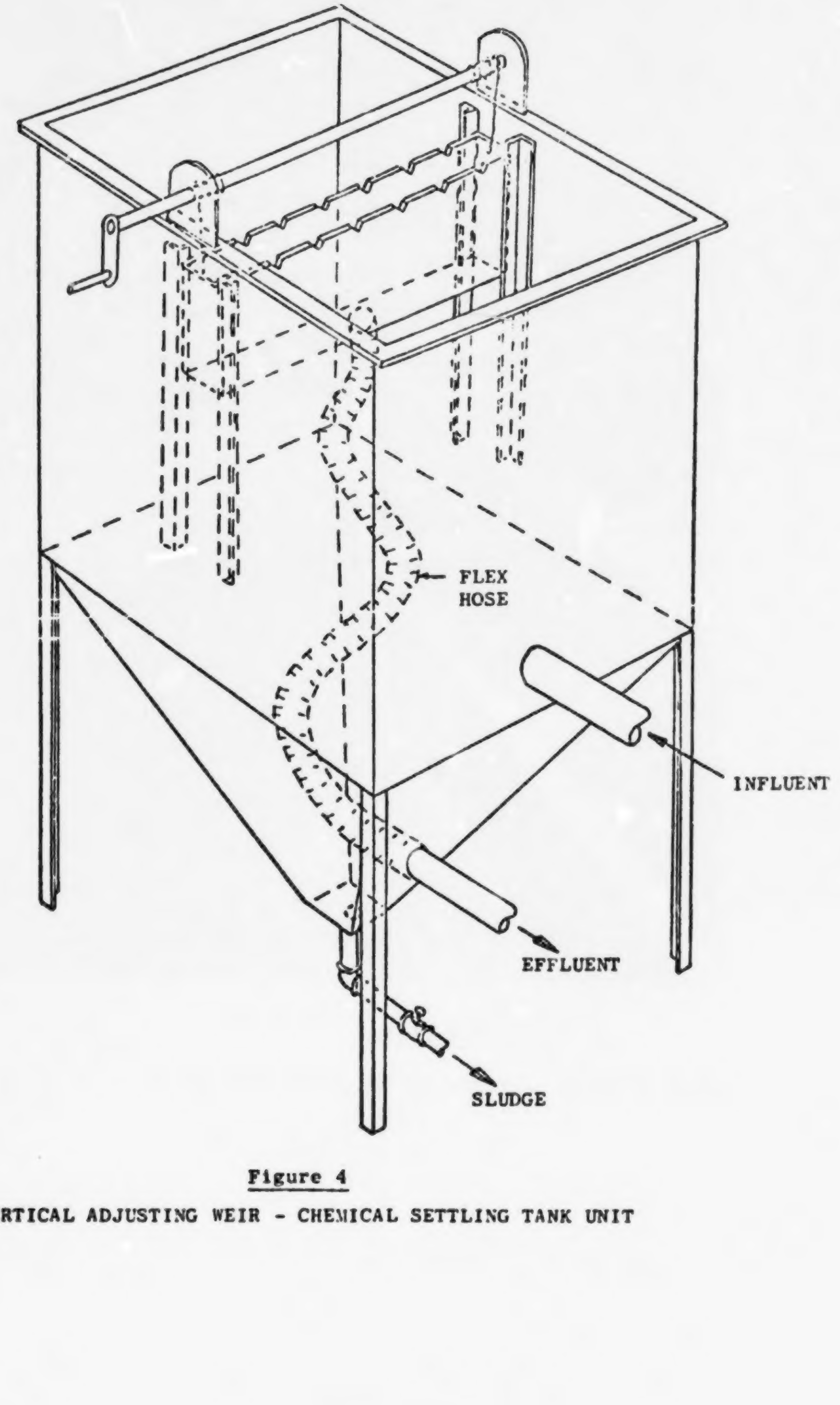
FLEX
HOSE
INFLUENT
EFFLUENT
SLUDGE

**Figure 4**

VERTICAL ADJUSTING WEIR - CHEMICAL SETTLING TANK UNIT minimize short circuiting and scouring. For the same reason, the line from the aeration basin to the clarifier was vented to the atmosphere to provide for removal of entrained air.

Sludge removed from the process stream by the clarifier was recycled to the influent of the nitrification tank by a small centrifugal pump coupled to a 3/4" rubber hose. Recycle flow was controlled by incorporating a valved recycle loop between the pump intake and discharge.

The biological clarifier was 4'0" square on top and the sides extended vertically downward 2'8". Below this depth the walls formed an inverted pyramid and extended another 3'8" to the apex. As with the other system tanks, the construction material was 3/16" steel plate. A 4' V-notch overflow weir was located along the centerline of the tank. From the weir a 2" flexible hose carried the liquid discharge to a fitting in the side of the tank.

The effluent from the biological system was subjected to chemical treatment for removal of suspended solids, phosphates, and selected organic and inorganic constituents. As indicated earlier, this was achieved in one of two parallel clarification systems, conventional and high-rate, by splitting the biological clarifier effluent into two flow streams. The conventional system incorporated facilities for coagulant addition, rapid mixing, flocculation, settling, and rapid sand filtration. Provision was made for adding either lime or alum coagulant to the nitrified influent.

The coagulant was rapidly dispersed throughout the water volume in a rapid mix tank having a one minute hydraulic detention time. This tank was 8" wide, 14" long and 19½" deep with four vertical baffles. The first three baffles had space on alternate ends for development of an end-around flow pattern. The final baffle extended entirely across the tank width with a vertical adjustment to control head and detention within the tank. The flow passed over this baffle and into the outlet line.

Following rapid mix, the flow in the conventional system was again split into parallel systems prior to flocculation and settling. This was done to maximize operational flexibility within the plant. The discharge from the rapid mix tank flowed to a tee where the flow was divided between each of the duplicate slow mix-clarifier systems. Each side of the tee was fitted with a 2" globe valve for isolation of the individual systems. The flow rate distribution between the two systems was controlled by adjusting the weir height in each of the parallel flocculation tanks.

The slow mix tanks were built of 3/16" steel plate. They were 2'4" by 2'0" on top and 3'4" deep to provide a 30 minute hydraulic detention at design flow. Each tank had a V-notch overflow weir similar to that in the biological clarifier except that they were only 2'4" long. Each tank was stirred by a four bladed paddle with the blades being $2\frac{1}{2}$" by 22" long and located 7 inches from the axis of the stirrer to the center of the paddle. The speed of the stirrer was variable over the range 8-33 rpm. Flow over the weir passed through flexible tubing and out of the tank through a fitting on the side. In each system, the slow mix tank effluent passed through 2" PVC to the chemical clarifier. These clarifiers were similar in operation to the bio-clarifiers. Each clarifier was 3'0" square on top and had an overall height of $4'8\frac{1}{2}$". Chemical sludge removed from the process flow was pumped to a holding tank on a batch basis and then pumped out for disposal.

Following chemical clarification, facilities were provided for recarbonation of the effluent. In this operation, carbon dioxide or mineral acid was added to lower the pH and stabilize the water prior to applying it to a filter. This was done to prevent calcium carbonate encrustation of the filter media. The recarbonation tank was 2' x 2'4" x 3'4".

The parallel chemical treatment system employed high rate clarification and included units for coagulant addition, rapid mixing, flocculation, polymer addition, and direct dual-media filtration. The system was identical to the conventional system through the slow mix step except that parallel slow mix units were not provided. From the slow mix tank the flow entered the intake to the filter pumps.

The filter pumps were Teel 1P555 screw type positive displacement pumps. They were driven by variable speed motors using belt drives. As with the nitrifier sludge recycle, these pumps were equipped with a recycle loop to provide control over the filter loading rates. There were two pumps installed with adequate piping to permit simultaneous pumping from each of two tanks, onto either of the two filters.

From the filter pump the process water flowed through 3/4" rubber hose to either the rapid sand or dual media filters. The rapid sand filter was $13\frac{1}{4}$ in. inside diameter and 6'0" tall. A plexiglass window was installed on the side of the filter which extended from 6" below the top. At each end of the filter there was a ¼" x 3½" flange to which ¼" endplates were secured. These endplates contained the fittings necessary for loading, discharging and backwashing the filters. A filter head loss fitting was provided on the side of each filter 1'6" down from the top of the filter. There were 7 ¼" x ¼" x 13½" annular rings welded to the inside of each filter starting at 6" above the bottom and placed every 4" upward. These were included to minimize the wall effects within the filters. There were two perforated baffles 3" apart in the bottom of each filter. Effluent flowed through a threaded fitting on the bottom of the filter and into an effluent holding tank from which it was pumped from the plant. The dual-media filter construction was identical to that of the rapid sand filter except that it was 8" inside diameter. The gradations of filter media within these two filters are shown in Tables I and II.

# TABLE I

## RAPID SAND FILTER COMPOSITION

Diameter 14" I.D.

| Layer Depth | Material | Characteristics | |
|---|---|---|---|
| 24" | Filter Sand | E.S. | .40 mm - .45 mm |
| | | All passed | #30 U.S.S. Sieve |
| | | All retained | #50 U.S.S. Sieve |
| 2" | Coarse Sand | All passed | #16 U.S.S. Sieve |
| | | All retained | #30 U.S.S. Sieve |
| 1½" | Coarse Sand | All passed | # 8 U.S.S. Sieve |
| | | All retained | #16 U.S.S. Sieve |
| 2½" | Gravel | All passed | # 4 U.S.S. Sieve |
| | | All retained | # 8 U.S.S. Sieve |
| 1½" | Gravel | All passed | 1/4" Sq. Sieve |
| | | All retained | # 4 U.S.S. Sieve |
| 1½" | Gravel | All passed | 1/2" Sq. Sieve |
| | | All retained | 1/4" Sq. Sieve |
| 1" | Gravel | All passed | 3/4" Sq. Sieve |
| | | All retained | 1/2" Sq. Sieve |

## TABLE II

## DUAL-MEDIA FILTER COMPOSITION

Diameter 8 1/8" I.D.

| Layer Depth | Material | Characteristics | |
|---|---|---|---|
| 16" | Anthrafilt | S.G. | 1.55 |
| | | Porosity | .45 - .55 |
| | | E.S. | .90 mm - 1.0 mm |
| | | U.C. | 1.7 |
| | | All passed | #12 U.S.S. Sieve |
| | | All retained | #30 U.S.S. Sieve |
| 8" | Filter Sand | E.S. | .40 mm - .45 mm |
| | | All passed | #30 U.S.S. Sieve |
| | | All retained | #50 U.S.S. Sieve |
| 2" | Coarse Sand | All passed | #16 U.S.S. Sieve |
| | | All retained | #30 U.S.S. Sieve |
| 2" | Coarse Sand | All passed | # 8 U.S.S. Sieve |
| | | All retained | #16 U.S.S. Sieve |
| 1½" | Fine Gravel | All passed | # 4 U.S.S. Sieve |
| | | All retained | # 8 U.S.S. Sieve |
| 2" | Gravel | All passed | 1/4" Sq. Sieve |
| | | All retained | # 4 U.S.S. Sieve |
| 1½" | Gravel | All passed | 1/2" Sq. Sieve |
| | | All retained | 1/4" Sq. Sieve |

## V. ANALYTICAL METHODS

### INTRODUCTION

The process waters of the pilot plant were routinely analyzed for each of the following constituents:

Alkalinity
Aluminum
Calcium
Bacteriological Parameters (coliforms, fecal coliforms, fecal streptococcus)
Hardness
Nitrogen, Ammonia
Nitrogen, Nitrate
Nitrogen, Organic
Oxygen, Dissolved (D.O.)
Oxygen Demand, Biochemical (BOD)
Oxygen Demand, Chemical (COD)
pH
Phosphate, Ortho-
Phosphate, Total
Residue, Total Volatile & Fixed
Residue, Suspended (Nonfilterable), Volatile, & Fixed
Trace Metals (iron, copper, manganese, chromium, lead, zinc, cadmium, molybdenum)
Turbidity

In addition, certain of the above analyses were performed on biological and chemical sludges as well as waste, and recycle streams.

Samples were collected hourly at each sampling point in the system and composited over each run. BOD samples were refrigerated prior to preparing them for incubation. Samples for ammonia, organic nitrogen, nitrate, phosphate, alkalinity, calcium, and total hardness were preserved by adding 40 mg of $HgCl_2$ to each liter of sample. Samples for COD and aluminum analyses were preserved with 2 ml./liter of concentrated $H_2SO_4$.

All of the techniques used in the analyses of pilot plant samples were exactly as described in the 12th and 13th Edition of Standard Methods for the Examination of Water and Wastewater (1)(2) except as noted below.

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## ALKALINITY

Alkalinity determinations were performed by potentiometric titration using a Radiometer Automatic Titrator. This apparatus titrated each sample and traced the titration curve. This curve indicated the volumes of acid titrant necessary to reach pH 8.3 and 4.5, respectively. Alkalinity was calculated from this information as in the standard colorimetric or potentiometric tests described in the 13th Edition of Standard Methods. The use of the automatic titrator removed errors due to the different perception of end-point color change among different individuals.

## BACTERIOLOGICAL ANALYSES

Tests were run for coliforms, fecal coliforms, and fecal streptococcol bacteria. The analyses were performed by the Denver Water Department laboratories using the Membrane Filter Technique described in the 13th Edition of Standard Methods.

## CALCIUM

The EDTA titriometric method was used as described in the 13th Edition of Standard Methods. The modification of section 3a. was practiced as all calcium samples were previously titrated in the course of the alkalinity determination and then heated to incipient boiling for 30 minutes.

## HARDNESS

This analysis was performed by the EDTA titrimetric method described in the 13th Edition of Standard Methods (No. 122.B.).

## NITROGEN, AMMONIA

This test was performed as described in the 13th Edition of Standard Methods (No. 212).

## NITROGEN, NITRATE

The Brucine Method presented in the 12th Edition of Standard Methods (p. 198-200) was used without modification except that samples were pretreated by centrifugation to remove suspended matter.

## NITROGEN, ORGANIC

This analysis was performed exactly as described in the 13th Edition of Standard Methods (No. 215). The samples tested were the same as those previously used for the ammonia determination.

## OXYGEN, DISSOLVED

Dissolved oxygen was determined both by the azide modification of the Iodometric Method as described in the 13th Edition of Standard Methods (No. 218.B), and by the Membrane Electrode Method (No. 218.F). For the membrane electrode method a Yellow Springs Instrument Co. Model 54 dissolved oxygen meter was used. This meter was calibrated with the Iodometric D.O. determination.

## OXYGEN DEMAND, BIOCHEMICAL

This analysis was performed as described in the 13th Edition of Standard Methods (No. 219). All D.O. values were determined by the Iodometric method.

## OXYGEN DEMAND, CHEMICAL

COD analyses were performed by the method described in the 13th Edition of Standard Methods (No. 220).

## pH

Commercial pH meters were used with regular standardization against commercial buffer preparations.

## PHOSPHATE, ORTHO-

The phosphate analysis was modified somewhat to give better performance at the combination of high turbidity and color, and low phosphate concentrations found at some points in the plant. For low levels of phosphate characteristic of the effluent streams of the pilot plant the Stannous Chloride Method was used (Standard Methods, 13th Edition, No. 223.E). The modification consisted of the inclusion of separate blanks for each sample to eliminate effects of color and turbidity on the colorimetric determination.

Initially, each sample was homogenized by vigorous stirring in a blender for 5 minutes. After homogenization, a 25.0 or 50.0 ml. sample was pipetted into a clean 100 ml. volumetric flask. The samples were neutralized to the phenolpthalein endpoint and then diluted to 100.0 ml. with de-ionized water. Using a pipet, 50.0 ml. of sample was drawn off and placed in a clean, dry erlenmeyer flask. Ammonium molybdate solution (S.M. 13th Edition, No. 223.E.3.c) was then added to each of the samples in the erlenmeyer flasks. Strong acid solution was added to the corresponsing sample in the volumetric flask (Standard Methods, 13th Edition, No. 223.E.3.b). Stannous chloride reagent was added to all of the samples in the volumetric flask (Standard Methods, 13th Edition, No. 223.E.3.d). Thus, the samples in the volumetric flasks served as individual blanks for each sample. No color development occured as no ammonium molybdate was present. Stannous chloride reagent was added to each of the samples in the erlenmeyer flasks at intervals of one minute. After 11 minutes of development time, the transmittance of each sample was read against its corresponding blank. In this way 11 samples could be run simultaneously. It should be noted that the sample size after dilution was 50 ml. necessitating only one-half the prescribed volume of reagents (2.0 ml. of ammonium molybdate and strong acid, and 0.25 ml. of stannous chloride).

For higher phosphate levels an identical procedure was used, employing the Aminonapthylsulfonic Acid Method as described in the 12th Edition of Standard Methods (p. 231-234). The only difference between the two methods, other than the difference in reagents, was the shorter color development time of five minutes.

## PHOSPHATE, TOTAL

The Persulfate Digestion Method was used (13th Edition of Standard Methods, No. 223.C.III) to release phosphates from combination with organic matter. This digestion was integrated into the orthophosphate determination in the following manner. After samples were placed in the volumetric flasks and neutralized to the phenolpthalein end point, 0.75 grams of potassium persulfate was added to each sample. The samples were heated to incipient boiling for 105 minutes. After digestion, the samples were cooled and titrated with 3N NaOH to the pink color of phenolpthalein. The pink color was removed with dropwise addition of strong acid. The samples were then diluted to 100 ml. and analyzed just as for orthophosphate.

## RESIDUE, TOTAL - VOLATILE AND FIXED

This analysis was performed exactly as described in the 13th Edition of Standard Methods (No. 224 a & B).

## RESIDUE, SUSPENDED (NONFILTERABLE), VOLATILE AND FIXED

A modified method for the analysis of suspended solids was used to obtain consistent, accurate, and rapid analyses. The analysis was similar to commonly used methods except that in the sample filtration a composite filter mat was used. This mat was composed of a glass fiber disk overlain with coarse asbestos.

To prepare the filter crucible, asbestos (medium fiber, acid washed) was placed in a beaker and washed thoroughly until only the coarse fibers remained. This asbestos was then washed in distilled or de-ionized water for 15 minutes and stored in a distilled water suspension.

The crucible used was a Gooch type Coors No. 4 or equivalent. The glass disk was 2.4 cm. in diameter, Reeve Angel #934AH or equivalent. The disk was placed in the bottom of the crucible and suction was applied. Washed asbestos was added to the crucible, suspending fibers with distilled water during the addition. This was continued until a 1/16-1/8" asbestos mat was developed. The edges were tamped and the mats were allowed to dry prior to firing the crucibles at 600°C for 20 minutes. The fired crucibles were cooled and stored in a dissicator for subsequent use.

The samples were introduced in such a way as to keep the mat submerged during filtration. Care was taken to avoid disturbing the mat during sample filtration. After sample filtration, the sample container was triple rinsed with de-ionized water and the filter mat was rinsed with 10 ml. of de-ionized water.

The crucibles were dryed at 103° in a mechanical convection oven until the weight loss was negligible. After drying, the crucible was placed in a dessicator and cooled before weighing. Ignition of the crucible for determination of volatile and fixed solids was accomplished as described in the 13th Edition of Standard Methods (No. 224 D). Calculation of results for both total suspended solids, and the volatile and fixed fractions was performed as described in Standard Methods.

## TRACE METALS

The Denver Water Department laboratories provided heavy metal analyses for each of the following metals:

Iron
Manganese
Copper
Chromium
Lead
Zinc
Cadmium
Molybdenum
Mercury

The analytical technique used in each case was the Atomic Absorption Spectrophotometry Method. A Varian Techtron AA5 was employed with analytical procedures outlined in *Analysis of Trace Elements in Natural Waters by Atomic Absorption Spectrophotometry* (3).

## TURBIDITY

Turbidity measurements were made with a Hellige Turbidimeter.

# VI. BIOLOGICAL NITRIFICATION

## INTRODUCTION

Nitrogen compounds are recognized as significant constituents in natural waters because of their demonstrated role as eutrophic stimulants, and for their toxic properties with respect to fish and humans. Since wastewaters provide a significant source of the undesirable aqueous nitrogen, removal of nitrogen compounds is indicated for tertiary treatment of wastewaters. In this water renovation study, conversion of the corrosive ammonia form nitrogen to the nitrate form was considered sufficient for the specific first-stage application of wastewater reuse for industrial purposes. In this context, the major purpose of this phase of the research program was to evaluate the important design parameters for biological nitrification. As indicated, nitrification is necessary for industrial reuse because of the corrosive nature of ammonia. In a solution of ammonia complexants containing free available oxygen, copper corrodes rapidly with the formation of the complex $Cu(NH_3)_4^{++}$ ion. The rate of corrosion is directly related to the amount of ammonia present in the water. Ammonia also causes season cracking in brass. It has been shown that an ammonia concentration of 30 ppm in steam condensate leads to deep grooving in unstressed brass, and stress corrosion cracking in stressed brass (4 ). The oxidized form of ammonia is the nitrate ion ($NO_3^-$) which is stable and does not react in a corrosive manner with pipes, fittings, and process machinery. A subsequent investigation must address the problem of complete nitrogen removal when higher level reuse applications are considered.

## AEROBIC OXIDATION

In the biological nitrification process, ammonia is oxidized to nitrate by autotrophic bacteria. These bacteria fix carbon dioxide as a source of carbon for cell material, and obtain energy from the process by oxidizing inorganic substrates. Two groups of autotrophic bacteria are distinguished in nitrification and each is responsible for a specific phase of the process. The group Nitrosomonas oxidizes ammonia to nitrites. The group Nitrobacter oxidizes nitrites to nitrates. The two step process is expressed as follows:

Nitrosomonas: $NH_4^+ + 1.5\ O_2 \rightarrow NO_2^- + 2H^+ + H_2O$

Nitrobacter: $NO_2^- + 0.5\ O_2 \rightarrow NO_3^-$

As indicated by Stankewich ( 5 ), the stoichiometric demand for oxygen would be 4.57 lbs. $O_2$/lb. $NH_3$-N. However, as was shown theoretically and experimentally by Stankewich some of the ammonia is taken up in cell synthesis thereby reducing the oxygen requirement to approximately 4.33 lb. $O_2$/lb. $NH_3$-N. With a very short sludge age the oxygen requirement would be less, and should range between 4.0 and 4.6.

One gram of substrate metabolized by the autotrophs provides the following energy yield ( 6 ):

Nitrosomonas: ammonia → nitrite + 2 kcal/gram

Nitrobacter: nitrite → nitrate + 0.3 kcal/gram

The ammonium compound metabolized by the autotrophic nitrifier provides energy to the microorganism through an electron transfer system coordinated by coenzymes found in the microbial cell. The electron transfer system involves the removal of hydrogen during metabolism with the effect that the ammonium compound is oxidized. The best indicator of the expenditure of biological energy is the oxygen utilized under aerobic conditions. Oxygen is used to regenerate DPN + $H_2$ (coenzyme diphosphopyridine nucleotide with attached hydrogens) and ammonia is oxidized according to the preceeding equation with subsequent removal of hydrogen by DPN. The oxygen uptake is then a function of both the hydrogen removal and the ammonia oxidation, two major sources of biological energy.

As has been implied, a source of free oxygen is required in such a bacterial system. This is generally achieved in an aeration tank where the biomass and the wastewater are mixed and aerated by air bubbles from a compressed air source.

## Nitrification in Secondary Sewage Treatment Plants

In conventional sewage treatment heterotrophic bacteria oxidize organic carbon and receive both food and energy from this source. The heterotrophs of the conventional activated sludge treatment grow at a much faster rate than the autotrophic nitrifiers. This is significant since the growth rate of a bacterial group has a predominant effect on the ratio of that group to the total bacterial population in a mixed culture such as encountered in the sewage treatment process. For steady state conditions at a particular biomass concentration and feed rate containing sufficient food a fas growing type of bacteria, such as the heterotrophs, will predominate over the slow growing types. This is particularly true for relatively short activated sludge aeration periods since the excess cells produced must be wasted in order to keep a constant biomass concentration in the aeration tank. The slow growing nitrifying bacteria will be wasted along with the heterotrophs, with the result that the nitrifiers will rarely develop a sufficient number in the aeration tank for significant ammonia oxidation. If, however, the aeration time is increased, two things occur. First, the nitrifiers have a longer period of time in which to reproduce so that their numbers increase if enough substrate ammonia is present. Second, the endogenous phase which develops for the heterotrophs permits ammonia oxidation by the nitrifying bacteria. A recent study of nitrogen balances on five existing activated sludge and trickling filter sewage treatment plants has shown that nitrification can and does occur to varying degrees in conventional plants (7). The observed oxidation was erratic and did not correlate to carbon or suspended solids removals. The study also showed that partial denitrification accompanied biological nitrification.

### Two-Stage Activated Sludge

Another means of limiting heterotrophic growth and maintaining nitrifiers in a system is to keep the influent food to the activated sludge in a ratio which constrains the metabolic activity of the heterotrophs to the endogenous phase throughout the aeration period. This can be done by using a tertiary activated sludge system acclimated for biological nitrification (5) (8). This principle is applied in the pilot plant studies described in this report. The majority of the organic constituents in the influent to the experimental system have been oxidized in the preceding secondary treatment so that the incoming food concentration is low. In this situation, the nitrifiers continuously circulate in the system with little net growth, and oxidize all of the wastewater ammonia provided they have adequate detention and oxygen. Barth, Brenner and Lewis (8) have shown such a system to produce about 90% nitrification.

## PROCESS CONTROL PARAMETERS

Previous investigations have identified several important parameters in the design and control of nitrification process operation. Included among the most significant of these parameters are the mixed liquor solids concentration, organic and substrate loadings, sludge settling characteristics, reaction time, recycle rates, temperature, pH, and oxygen utilization coefficients. These variables were studied in this investigation for the specific purpose of applying the results to the Denver situation.

### Mixed Liquor Suspended Solids

The aeration tank contains a mixture of suspended and dissolved solids. The dissolved solids include soluble organic and inorganic compounds, and consist mostly of salts. The suspended solids are made up of a suspension of bacteria and solid inorganic and organic particles in the mixed liquor. The volatile fraction of the suspended solids is that portion that volatilizes after exposure to 600°C temperatures. This includes most organic substances which would make up the active bacterial mass. The percent of volatile solids in activated sludge is usually from 70 to 80 percent. The concentration of MLSS in the aeration tank determines the organic loading rate when the wastewater flow and strength is constant.

### Organic and Substrate Loading

The organic load is expressed as lb. BOD per day per lb. mixed liquor suspended solids. This loading parameter together with the reaction time has been shown to directly effect the efficiency of the nitrification process. As was mentioned, when the carbonaceous content of the influent becomes large for a unit of time, the growth of heterotrophic bacteria increases to a point where wasting of sludge becomes necessary. It is at this point that the nitrifying autotrophs are lost to the system. The substrate load, in this case ammonia, is given in the same terms as the organic load, lb. $NH_3$ per day per lb. MLSS. The loading values can be controlled through hydraulic loading or by variation of the MLSS.

### Settling Characteristics

The settling characteristics of biological sludges are typically represented by the Sludge Volume Index or SVI. The Sludge Volume Index is an indication of the density of the sludge. The SVI is defined as the volume occupied by the sludge fraction in one liter of mixed liquor after 30 minutes of settling, divided by the dry weight concentration of the sludge.

The SVI has inverse dimensions of density (ml/gm). Values of SVI from 50 to 100 indicate a good settling sludge, while values over 200 indicate that the sludge is light, bulked, and hard to settle.

## Recycle Rate

Recycling of suspended solids from the clarifier back to the aeration tank serves to hold the bacteria in the system, thereby providing a means of controlling the MLSS concentration. If it is assumed that there is negligible net growth of bacteria in the system, and the recycle sludge concentration is one percent, the sludge recycle ratio can be related to the SVI and MLSS by the following equation:

$$\frac{R}{Q} = \frac{1}{\frac{100}{SVI \cdot \%MLSS} - 1}$$

In this relationship, R is the recycle flow rate, and % MLSS is the dry weight percent of suspended solids in the mixed liquor.

## Oxygen Utilization Coefficient

The oxygen utilization coefficient, micro-liters of oxygen per mg of MLSS per hour, expresses the oxygen requirements of a unit of biomass. This value can be found for a range of substrate loadings, temperatures, pH values, and other conditions. The oxygen utilization rate is essential in design to provide adequate aeration facilities. During actual plant operation the air flow rate can be controlled by the operator to give the proper dissolved oxygen levels at minimum compressor power costs.

## Reaction Time and Overflow Rate

Due to the design of the pilot plant in this study, it was not possible to vary the hydraulic load without proportionally varying the organic load for a constant MLSS concentration. This was true because the organic contents of the influent did not vary markedly from day to day. Increasing the hydraulic load decreased the available reaction time for bacterial substrate oxidation. Since flow into the plant was fairly constant, the effect of increasing the hydraulic load could be achieved by reducing the size of the units. By use of several sectioned units in series, it was possible to study several detention times concurrently.

Hydraulic load also effects the settling capabilities in the clarifier and in many cases the settling characteristics of the sludge. Although guidelines have been established for overflow rates of clarifiers, they do not apply consistently to situations involving activated sludge variations. The conditions in the aeration tank determine the settling characteristics of the sludge, but the hydraulics of the clarifier need to be carefully controlled to make use of any positive settling characteristics possessed by the sludge. Common design criteria require that overflow rates on activated sludge clarifiers not exceed 800 gpd/sq. ft. for average daily flow. Since nitrifying sludge nourished by secondary effluent may have somewhat different characteristics than conventional activated sludge, the maximum overflow rate may have to be decreased for good settling. The values of the parameters for which the nitrification system was designed are shown in Table III.

## EXPERIMENTAL PROGRAM

The nitrification studies included both bench scale and pilot plant investigations. The bench scale studies were made initially to provide direction in the operation of the pilot plant. These laboratory investigations involved batch fed four liter reactors. Parameters investigated in this work included detention time, oxygen uptake rates, loading variations, and the effects of variable temperatures on the performance of the nitrification system. A wide range of temperatures was studied in the bench scale tests because of the difficulty in varying this parameter at the pilot plant scale.

After initiation of the pilot plant studies, the plant was operated on a continuous basis with composite sampling of the influent and effluent streams three days each week. The influent and effluent samples were analyzed for $BOD_5$, COD, $NH_4$-N, organic nitrogen, $NO_2$-N, $NO_3$-N, turbidity, total solids, TSS, VSS. Additionally, the mixed liquor was sampled and analyzed for MLSS, MLVSS, pH, D.O., temperature, and SVI.

Two different aeration systems were used in this study to permit evaluation of a wide range of reaction times. One system incorporated six 50 gallon reactors in series. Through analyses of the effluent from each vessel in this series detention times of 12-72 minutes were evaluated at full flow. The other aeration system consisted of a single baffled 900 gallon tank. By adjusting the height of the discharge weir in this tank reaction

## TABLE III

## NITRIFICATION DESIGN PARAMETERS

| Parameter or Dimension | Value |
|---|---|
| Design Flow | 5 gallons per minute |
| Aeration Tank Detention Time | 0.2*-3.6 hours |
| Aeration Tank Volume | 7-145 cubic feet |
| Clarifier Detention Time | 1.6 hours |
| Clarifier Overflow Rate | 450 gallons per day per sq. ft. |
| Recycle Capacity | 100 per cent |

*for one 55 gallon drum reactor

times of 2-3.6 hours were investigated. Longer detention times were possible through diversion of a portion of the inflow. Coupled to these reaction time variations, the MLSS concentration was varied from 200-4000 mg/l in increments of approximately 500 mg/l. In this way, the effect of $BOD_5$, COD, and nitrogen loading rates could be assessed.

Aeration was provided with diffused air to maintain a dissolved oxygen concentration of 2 mg/l or greater. Other investigators have found that this level is sufficient with no substantial improvement in process performance at higher levels (5)(9)(10)(11)(12)(13)(14)(15). Likewise, the system pH was consistently in an acceptable range for nitrification. The pilot plant influent ranged between 6.8-7.8, with the majority of flows in the range 7.1-7.4. This is consistent with the optimum environmental conditions established by other investigators for Nitrosomonas and Nitrobacter (5)(9)(16)(17)(18)(19)(20).

Start-up of the nitrifying activated sludge unit required an extended period for growth of a nitrifying culture. To expedite this process, the units in this study were initially seeded with a sludge taken from aerobic digesters at the Metropolitan Denver Wastewater Treatment Plant. This sludge had received approximately eight days of digestion after being wasted from the secondary process. With this seed sludge, the pilot plant normally developed full nitrification capability within about one week. Using no seed material Mechalas, Allen and Matyskiela (21) found that 6-8 days of acclimation was required before complete nitrification was achieved in their submerged ring unit. Haug and McCarty (16) reported an acclimation time requirement of four months in the submerged filter configuration.

## EXPERIMENTAL RESULTS

### Temperature Effects

The initial bench scale studies were made to establish the effects of variable temperature on the rate and degree of nitrification. Active cultures of nitrifying activated sludge seed were obtained from an extended aeration treatment plant. Trickling filter effluent was used as the substrate in the four liter fill and draw units. The temperature effects on nitrification at a constant loading rate are shown in Figure 5. From these data it can be noted that for acclimated organisms at an average MLSS concentration of 780 mg/l, with reaction times between 4 and 8 hours, nitrification was nearly complete over the entire 5-30°C temperature

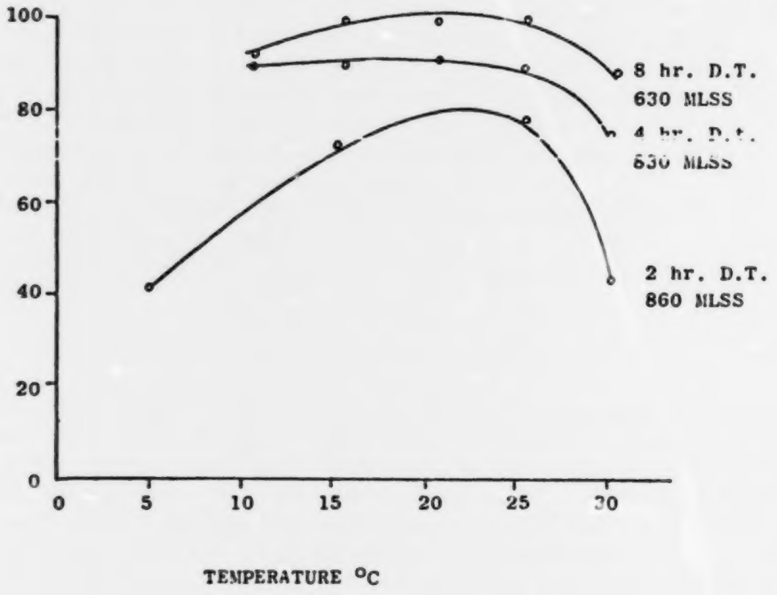

Figure 5

EFFECT OF TEMPERATURE ON NITRIFICATION FROM BENCH SCALE STUDIES range. With a 2 hour detention time, the nitrification efficiency decreased markedly. The maximum efficiency of about 80% was achieved between 20-25°C. A similar efficiency maximum was reported by Sawyer and Rohlich (22). Other authors have observed similar increases in nitrification rates for temperatures up to 25°C (9)(16).

The effect of aeration temperature on sludge settleability was also investigated in this study. Using the SVI as an indicator of the sludge settling characteristics, it can be seen in Figure 6 that the settleability improved with increasing temperatures. Sawyer reported a somewhat similar result (23).

## BOD Loading

Preliminary bench scale studies were also run to assess the effect of BOD loading on the efficiency of the nitrification process. The results of this early investigation are summarized in Figure 7. These results were confirmed by the pilot plant data presented in Figure 8. The scatter of pilot plant data on this figure apparently occurred because of the relatively frequent changes in loading conditions with the result that organism acclimation was not always complete. It can be noted that complete ammonia nitrification appears to occur up to a $BOD_5$ biomass loading of approximately 0.4 #/#/day. There are also indications from this plant data that the loading rate might be extended slightly higher with experience in operation. Several other investigators have found the maximum loading rate to be in a similar range as shown in Table IV.

## COD Loading

The degree of ammonia nitrification as a function of the COD loading is indicated by the data in Figure 9. The tendencies identified in the previous BOD loading curve are also evident for COD. The maximum COD loading for complete ammonia conversion appears to be approximately 1.4 #COD/#MLSS/day. This is in general agreement with Mechalas, Allen, and Matyskiela (21) who found that nitrification efficiency was not adversely affected below a load factor of 1.1 #COD/#MLVSS/day.

## Ammonia Loading

The ammonia oxidation as a function of the ammonia loading factor is shown in Figure 10. It can be noted that a loading of 0.16 #$NH_4$-N/#MLSS/day was found to be the upper limit for

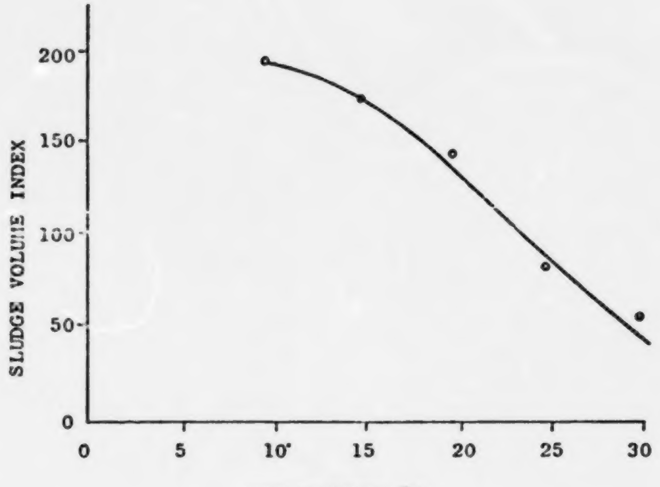

Figure 6
SLUDGE VOLUME INDEX AS A FUNCTION OF TEMPERATURE FOR THE BENCH SCALE STUDIES UTILIZING A FOUR HOUR DETENTION TIME

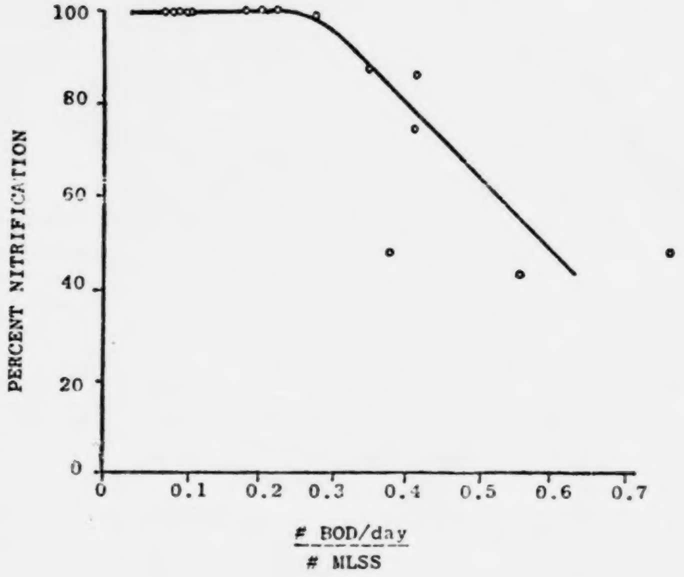

Figure 7

EFFECT OF BOD LOADING ON NITRIFICATION FROM BENCH SCALE STUDIES

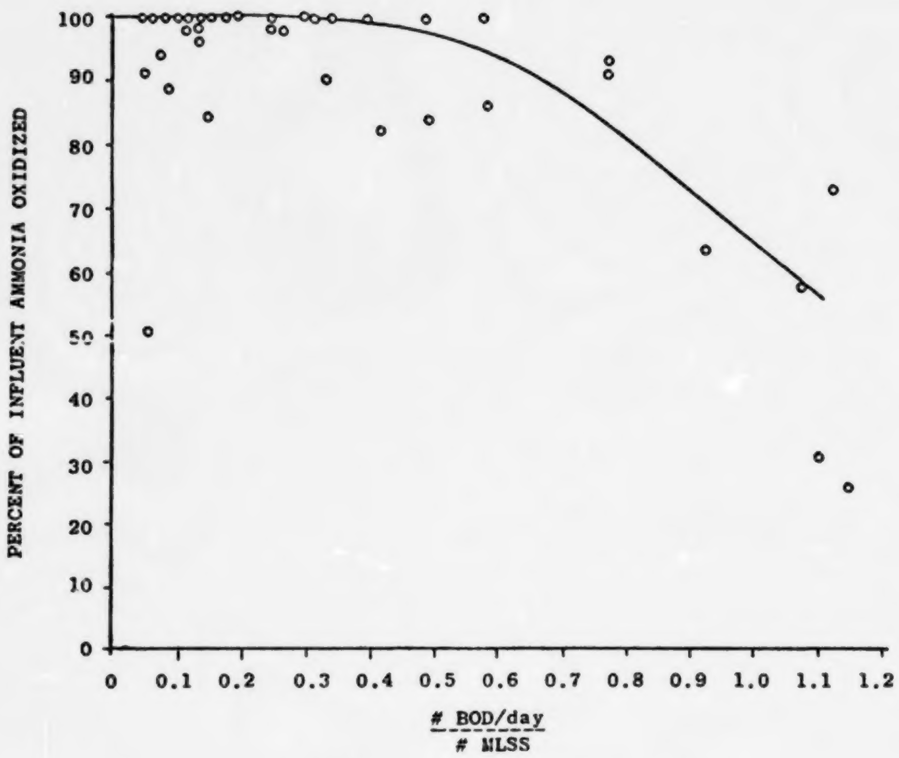

Figure 8

AMMONIA OXIDATION AS A FUNCTION OF BOD LOADING FACTOR

## TABLE IV.

### Comparison of BOD Loading Data

| Maximum $\frac{\#BOD/DAY}{\#MLSS}$ | Minimum Sludge Age-Days | Reference | Comments |
|---|---|---|---|
| 0.3-0.4 | 3-4 | (10) | Operated at 2800 mg/1 MLSS and a D.T. of 8 hours. |
| 0.25-0.33 | 4 | (24) | |
| 0.25-0.35 | - | (25) | |
| 0.28-0.4 | 5 | (5) | Pure Oxygen |

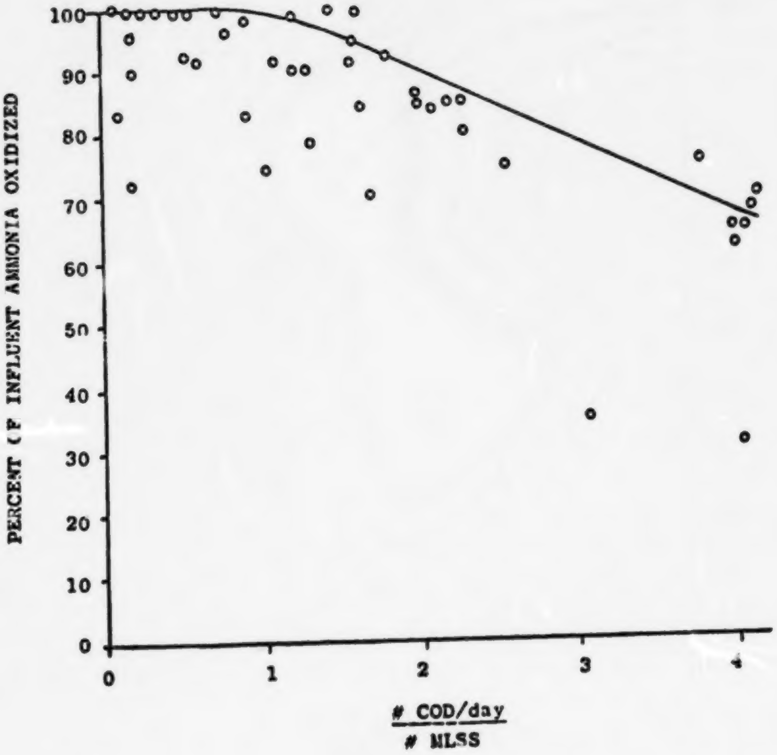

Figure 9

AMMONIA OXIDATION AS A FUNCTION OF THE COD LOADING FACTOR

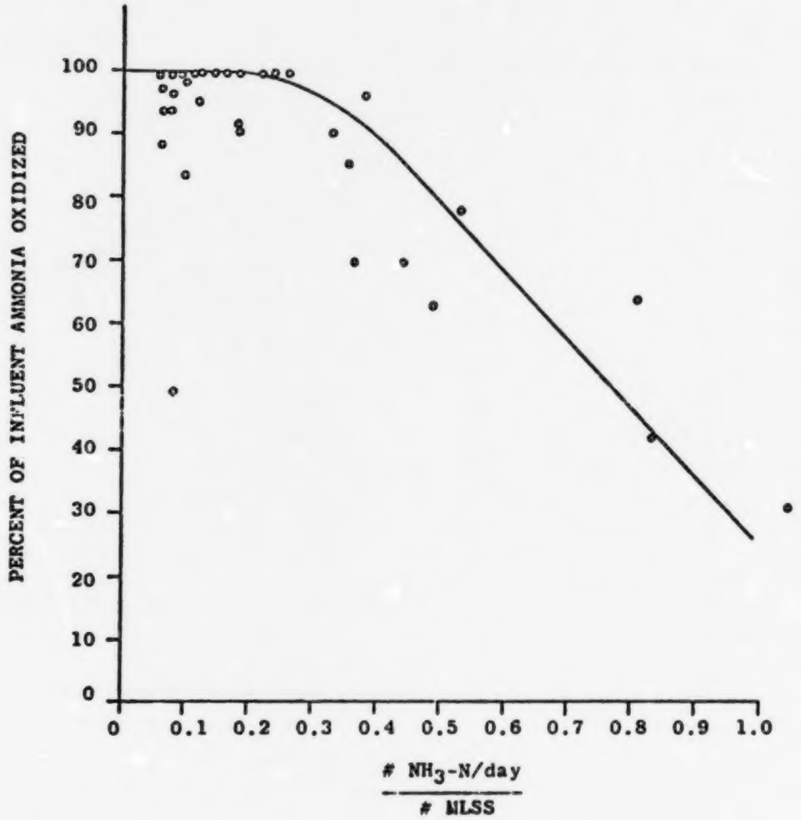

Figure 10

AMMONIA OXIDATION AS A FUNCTION OF AMMONIA LOADING FACTOR

FROM PILOT PLANT STUDIES

complete oxidation. For activated sludge containing approximately 82 percent volatile matter this would be equivalent to 8 mg $NH_3$/Hr/g. MLVSS. This value is slightly higher than the value of 7.7 determined by Wild, Sawyer, and McMahon ( 9), and 5.2 established by Mechalas, Allen, and Matyskiela (21).

## Sludge Settling

The activated sludge clarifier used in this study was designed with an overflow rate of 450 gpd/$Ft^2$. Even at this relatively low loading rate some solids carry-over was experienced. The nitrifying sludge is very light, due to the fact that it contains a very high fraction of bacterial mass with very little inert weighting matter. For this reason, it appears that very low overflow rates will be necessary in design of nitrification system clarifiers.

Because of the difficulties in scaling down clarifiers to pilot plant size, it was not possible to develop quantitative settling tank design criteria. In general, the settling tank performance was satisfactory, although pinpoint floc did commonly carry-over at a suspended solids level of 30-40 mg/l. It was noted that carry-over was less pronounced in the pilot clarifier at MLSS concentrations above 2000 mg/l. In this range, a blanket formed in the lower portion of the clarifier which improved the solids capture opportunity. During most of the study, there was a very gradual reduction in the MLSS concentration due to the outflow in the clarifier effluent. This necessitated periodic supplementation of the nitrifying sludge with solids from the Denver aerobic digestion units. Jar tests were made with several polyelectrolyte compounds added to the inflow to aid in capturing the pinpoint floc. None of these was effective within the reasonable economic dose limits.

## Reaction Time

The data developed with the six 50 gallon aeration tanks was used to establish the relationship between the extent of ammonia nitrification and detention time. The six units were operated in series with the settling tank at the end of the series and the return sludge pumped back to the first tank. The water accompanying the return sludge was completely nitrified thereby introducing a substantial amount of nitrate in the first tank. For this reason it was necessary to subtract the recycle nitrate values from overall nitrate conversion values in each vessel. The results of this analysis are shown in Figure 11. It can be noted that the rate of oxidation is nearly linear indicating that this rate is essentially independent of ammonia concentration. This observation was also presented by Wild, et al (9).

A theoretical analysis of the effect of ammonia level on reaction rate can be developed from a biological kinetics point of view using the Michaelis-Menten equation:

$$v = V_m S/(K_m + S)$$

where v is the reaction velocity, $V_m$ is the maximum rate at which the reaction can occur, $K_m$ is the Michaelis constant and S is the substrate concentration. Michaelis constants are available (12) for the nitrification reactions. The Nitrosomonas step is the rate controlling reaction and has a reported value of $K_m$ of 0.8 ppm at 20°C. Using this value, the reaction velocity can be calculated at different substrate levels.

| S (mg/l) | v |
|---|---|
| 5 | $0.86\ V_m$ |
| 10 | $0.93\ V_m$ |
| 15 | $0.95\ V_m$ |

It can be seen from this approach that the reaction rate should be nearly constant and becomes only slightly reduced at lower substrate levels. Although the Michaelis-Menten equation should be used very cautiously with mixed cultures and sequential reactions of the nitrification process, the analysis does give some insight into the question.

It can also be noted from the data that there is a very close correlation between rate of nitrification and the reactor organism content, represented by the MLSS concentration. The average inflow ammonia concentration during this test series was 18.3 mg/l. Using this value with the data in Figure 11, a nitrification rate of 0.16 lb. $NH_3$/lb. MLSS is calculated from the following relationship:

$$\frac{18.3 \text{ mg/l } NH_3\text{-N} \times 1440 \text{ min./day}}{4000 \text{ mg/l MLSS} \times 40 \text{ min. DT}} = 0.16 \ \frac{\text{lb. } NH_3\text{-N}}{\text{lb. MLSS day}}$$

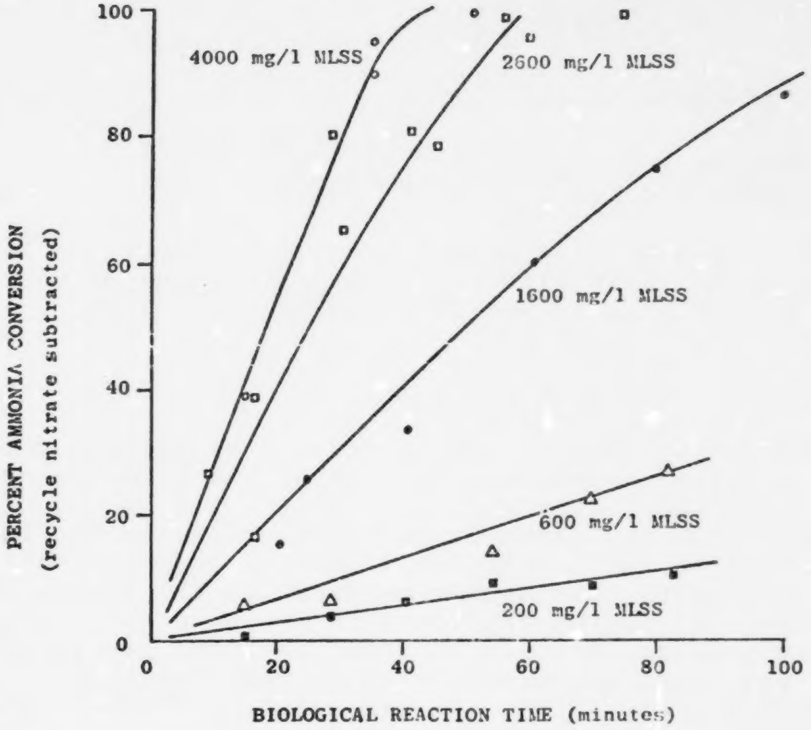

Figure 11

EXTENT OF BIOLOGICAL NITRIFICATION AS A FUNCTION OF REACTION TIME FROM PILOT PLANT STUDIES

This is based on complete nitrification in 40 minutes at 4000 mg/l, or in 100 minutes at 1600 mg/l.

The reaction can be approximated by the equation of the slope of the lines on the plot:

$$\% \text{ oxidation} = Kt$$

where: t = detention time in minutes = tank volume/inflow rate. K is shown to be directly proportional to MLSS and is assumed to be inversly proportional to ammonia concentration. Therefore:

$$\% \text{ oxidation} = \frac{\text{MLSS} \times t \text{ (min)}}{87 \times NH_3\text{-N (mg/l)}}$$

which can be rearranged to yield:

$$\begin{array}{c}\text{least detention time (hr.)} \\ \text{at 100\% oxidation}\end{array} = \frac{150 \times NH_3\text{-N (mg/l)}}{\text{MLSS}}$$

The above equations represent severe over-simplifications but can serve as a basis for making rough approximations of aeration tank sizes operating in the 15°C temperature range.

## Oxygen Utilization

The oxygen requirement of the nitrifying sludge was determined by making oxygen probe measurements of the rate of oxygen depletion in the mixed liquor from each of the six 50 gallon reactors. Samples of the mixed liquor were collected in BOD bottles and saturated with oxygen by injecting pure oxygen into the bottles. Following this sample preparation the dissolved oxygen concentrations in the bottles were measured and recorded at 30 second intervals. Results of these analyses are presented in Table V. From these measurements the oxygen utilization was found to be 0.91 $\#O_2$/day/#MLSS. As shown in the table, ammonia measurements were also made on the influent to each tank and it was found that ammonia oxidation was complete at the end of the third reactor. Oxygen utilization values for endogeneous respiration were determined from measurements of oxygen utilization in the last three reactors. This value was found to be 0.18 $\#O_2$/day/#MLSS. The difference between these two utilization rates represents the oxygen utilization for nitrification of 0.73 $\#O_2$/day/#MLSS. Based on the ammonia conversion value of 0.16 $\#NH_3$-N/day/#MLSS presented earlier, the oxygen demand for ammonia oxidation by nitrification was calculated to be 4.6

## TABLE V

### Oxygen Utilization for Nitrification

| Reactor | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|
| Ave. D.O. Depletion (mg/1/30 sec.) | 1.0 | 0.93 | 0.50/0.2 | 0.2 | 0.2 | 0.17 |
| Influent $NH_3$-N (with 120% recycle) | 6.0 | 3.8 | 2.9 | 0 | 0 | 0 |
| MLSS | 3252 | 3132 | 2972 | 3154 | 2971 | 3144 |
| MLVSS | 2478 | 2348 | 2166 | 2332 | 2174 | 2330 |

$\#O_2/\#NH_3$-N. This compares very closely with the stoichiometric quantity of 4.57 $\#O_2/\#NH_3$-N indicated by Stankewich (5).

Several qualitative observations can be made from the long term pilot plant operation. The nitrifying sludge was considerably more difficult to develop than normal secondary activated sludge. It did not settle as well and could be lost by gradual attrition. A source of sludge make-up from an aerobic digester is convenient and may be even necessary for continued long term operation.

In the oxidation of carbonaceous matter in the secondary process of activated sludge treatment the rate of oxidation changes throughout the oxidation period as the more easily assimilated materials are oxidized and only the more resistant materials remain. Complete oxidation never occurs because some of the carbonaceous compounds in sewage are too resistant to be oxidized within a reasonable treatment time. With nitrifying activated sludge, only a single substrate is involved. For this reason the rate of oxidation is nearly constant as long as the ammonia feed source exists and the extent of oxidation is complete if an adequate reaction time is provided.

# VII. CONVENTIONAL CLARIFICATION

## INTRODUCTION

Treatment systems employing chemical addition, flocculation, sedimentation, and filtration have long been used in the water supply field for removing suspended matter from raw water supply sources. Relatively recently this same treatment sequence has been adapted to problems of wastewater treatment. In this context, the impetus has developed from a recognized need to reduce the discharge of phosphorus to natural watercourses because of its demonstrated role in promoting excessive algal growth. Chemical precipitation has proven to be an effective means of accomplishing the desired phosphorus removals. Accompanying the concentration reduction of this algal nutrient, other classes of pollutants have also been removed in varying degrees. Thus, this type of treatment sequence has shown good potential for upgrading wastewater discharges to a quality level which will permit some levels of water reuse.

Conventional aluminum and iron coagulants, as well as lime, have been shown to be effective as chemical additives for upgrading wastewater effluent quality in this conventional clarification sequence. Of these coagulants, lime has received the most attention because of the potential for chemical recovery through recalcining of the lime sludge.

Lime reacts with the bicarbonate alkalinity in wastewater to form the calcium carbonate precipitate. It also reacts with orthophosphate to precipitate hydroxylapatite according to the following reaction:

$$5\ Ca^{++} + 4\ OH^{-} + 3\ HPO_4 \rightarrow Ca_5(OH)(PO_4)_3 + 3\ H_2O$$

Apatite is a crystalline precipitate with a Ca:P ratio reported to vary between 1.33-2 (26). The indicated variation in apatite composition is thought to result from the substitution of hydrogen ion for calcium within the crystal structure. The solubility of hydroxylapatite decreases rapidly with increasing pH. Schmid and McKinney (27) have shown that at pH 9.5 nearly all orthophosphate is converted to the solid precipitate. At wastewater pH levels above 9.5, magnesium hydroxide precipitation also occurs within the system. This gelatinous precipitate aids the physical removal of the fine hydroxylapatite precipitate as well as other suspended solids existing in wastewater.

## EXPERIMENTAL RESULTS

### Lime Precipitation

Throughout the chemical clarification studies, the expressed aim was to evaluate those parameters of particular importance in the design and operation of a plant scale system. In this regard, the following system variables were evaluated over a relatively broad range with subsequent analysis of the impact on the plant effectiveness:

1) lime dose
2) pH
3) flocculation intensity
4) flocculation detention time
5) filter loading rates
6) chemical sludge recycle.

Of course, parameters (1) and (2) above are interdependent and they relate to the optimal chemical environment within the system for removal of the constituents which were considered to be of most importance in this study. These included: phosphate, BOD, COD, solids, turbidity, bacteria, and heavy metals. Aside from item number 6, the other major system variables relate to variations of physical components within the system.

In the daily pilot plant evaluation, one-liter samples were composited over a four-hour period beginning after the system had reached equilibrium. The points sampled were 1) the pilot plant influent, 2) the bioclarifier effluent, 3) the clarifier effluent, 4) the recarbonation discharge, and 5) the filter effluent. While the samples were being composited, solids analysis (including suspended solids, suspended volatile solids, total solids, and total volatile solids), pH and temperature measurements, and turbidity determinations were made on samples from the five sample points. During the run, filter headloss measurements were recorded for the rapid sand filter. These were continued throughout the filter run or until headloss reached 14.5 feet, which was the limit for the system. Samples for trace metal and bacterial analyses were also collected during this time and later analyzed by the laboratory of the Denver Water Board.

The first set of studies was developed to define the impact of variations in the flocculator operation on the effectiveness of the clarification system. Both flocculation time and intensity were varied over the range permitted by the plant design. For the detention time studies, the mixing time was varied between 34-50 minutes while other plant variables were held constant. In the narrow range of detention times provided in this design there did not appear to be any significant difference in the plant performance. The consistency of performance is indicated in Figure 12, where total phosphates are plotted as a function of mixing time. Similar behavior was observed with the other wastewater constituents.

The mixing intensity in the flocculation basin was also studied under controlled conditions to isolate the effects of mixing rate variations on the clarification system performance. By varying the impeller speed in the flocculation basin, the mixing intensity represented by G, was investigated over the range 11-50 fps/ft. As shown in Figure 13, which is a plot of total phosphate vs. mixing intensity, G, variations of the flocculation intensity did not seem to alter the plant effectiveness in removing phosphates from wastewater.

Because of the strong pH dependence which has been shown for lime precipitation, one of the early studies was designed to evaluate the variation of pH with lime dose for the wastewater under investigation. The results of this portion of the pilot plant study are summarized in Figure 14. The lime dose variation covered the range of 0-600 mg/l $Ca(OH)_2$, with a corresponding variation in the wastewater pH following clarification of 7.1 to 11.7. With the alkalinity in this wastewater (250 mg/l) it was possible to achieve pH 11 at a moderate hydrated lime dose of approximately 275 mg/l. This is a desirable operational level because of the formation of gelatinous $Mg(OH)_2$ precipitate which aids in sweeping the fine precipitates from suspension.

Accompanying the variation in pH observed for variable lime doses was a significant variation in the effectiveness of lime clarification for removing the polluting constituents in wastewater. Data collected on turbidity and suspended solids reductions indicated that lime precipitation was very effective in reducing the concentration of particulate matter in wastewater. As shown in Figures 15 and 16, addition of 50 mg/l or more of lime with subsequent sedimentation and filtration resulted in significant reduction of the concentration of these constituents. More specifically, the turbidity was reduced from an average concentration of 17.5 mg/l $SiO_2$ to a level of consistently less than 3 mg/l. Likewise, the suspended solids were reduced from highly variable concentrations in the range 10-110 mg/l to consistently less than 1 mg/l. These levels of residual particulates are consistent with those reported by other authors using lime in somewhat similar clarification systems (28)(29). It should be noted from the figures that

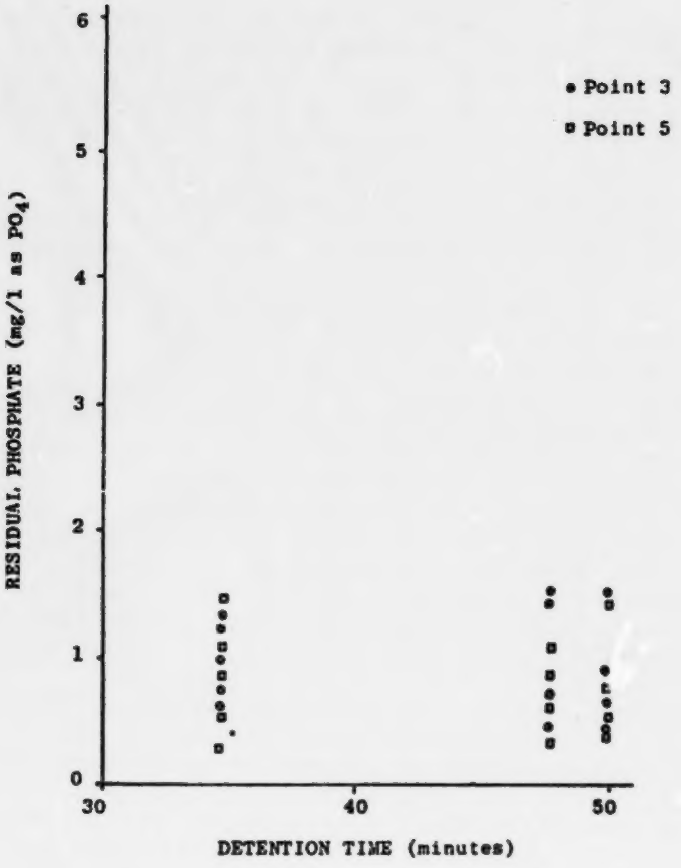

Figure 12

RESIDUAL PHOSPHATE AS A FUNCTION OF FLOCCULATION DETENTION TIME WITH ~ 300 mg/l LIME

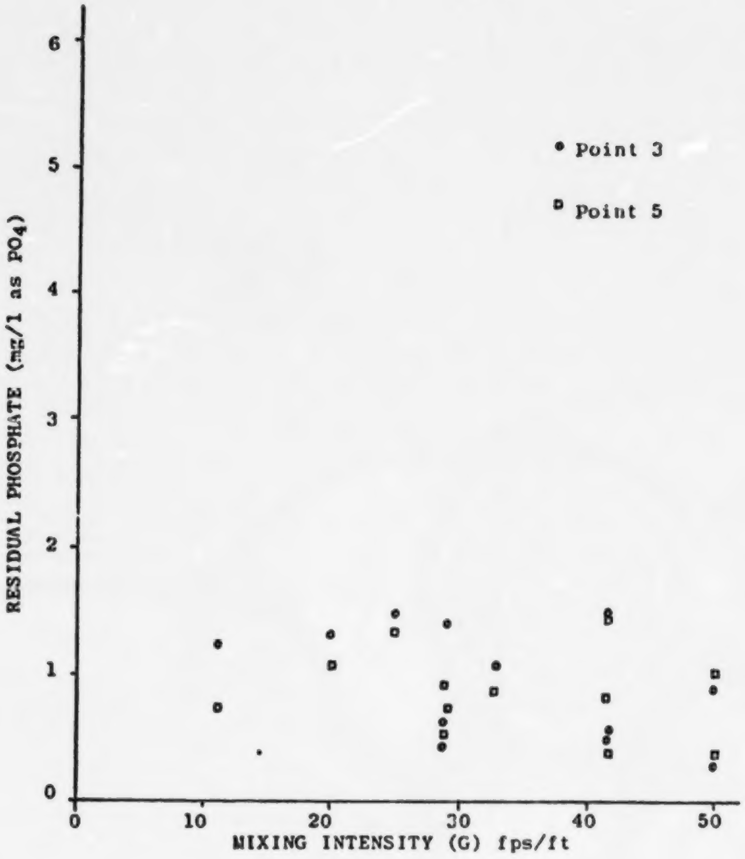

Figure 13

RESIDUAL PHOSPHATE AS A FUNCTION OF MIXING INTENSITY

WITH ~ 300 mg/l LIME

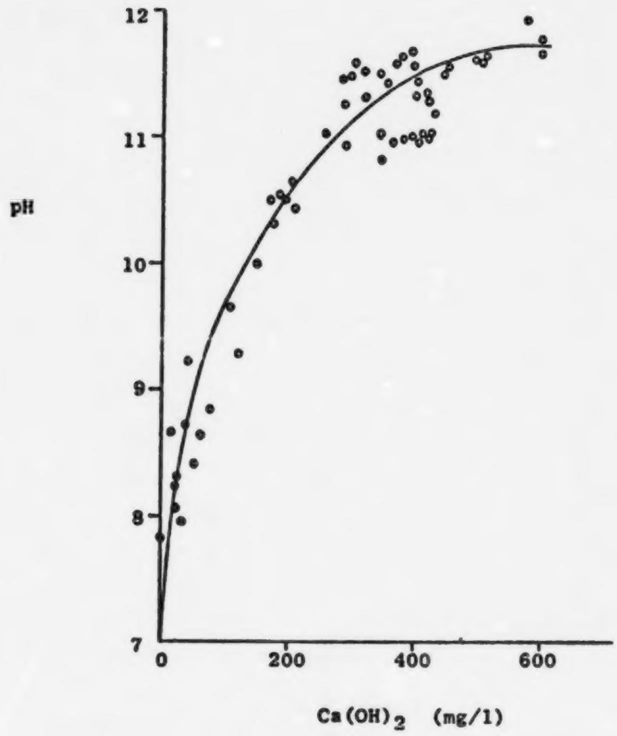

Figure 14
pH IN FLOCCULATION REACTOR AS A FUNCTION OF LIME DOSE

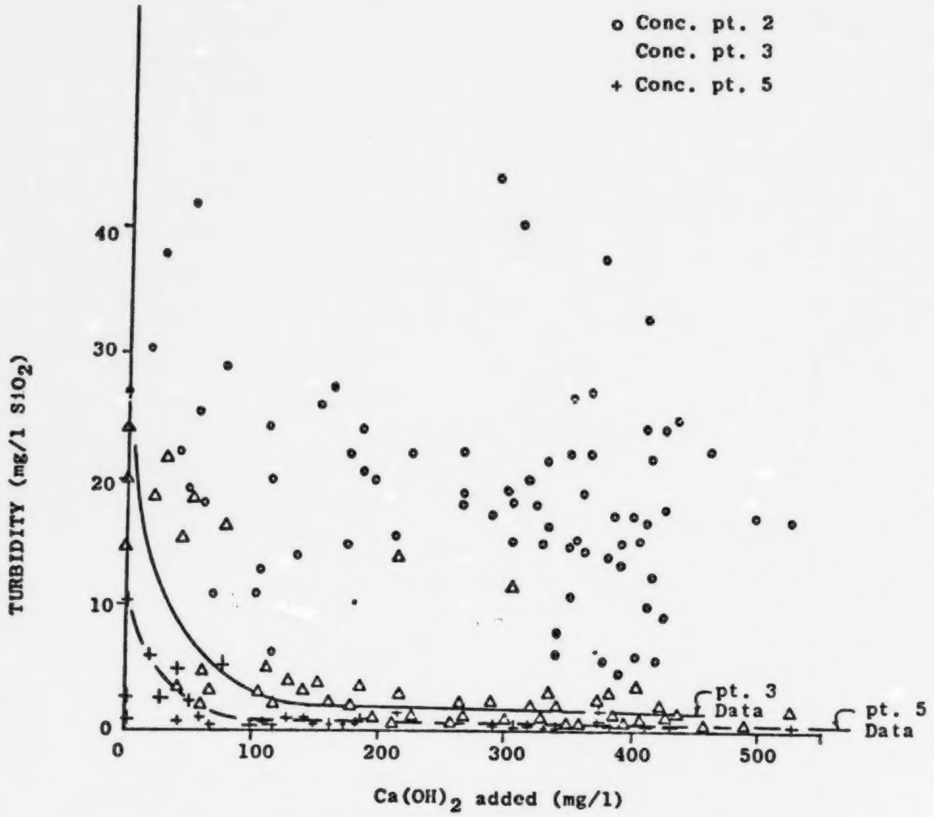

Figure 15

TURBIDITY REMOVALS WITH LIME ADDITION

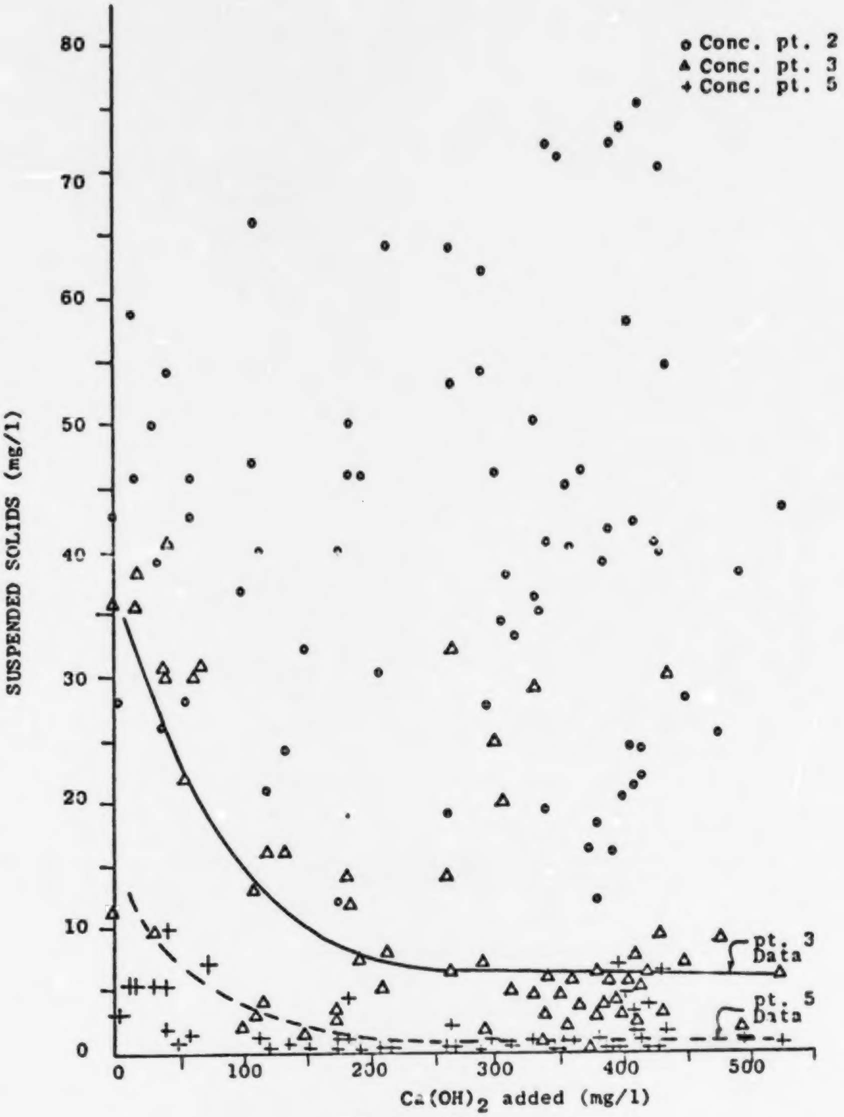

Figure 16

SUSPENDED SOLIDS REMOVAL WITH LIME ADDITIONS while most of the solids mass is removed in the settling system, the filter serves to remove approximately 50% of the solids in the clarifier over-flow and assures that a consistent high quality product is delivered in spite of any periodic solids carry-over which might occur in the settling system.

The response of BOD in the wastewater effluent to the lime dose applied in the clarification system is shown in Figure 17. BOD concentrations in the influent to the clarification system were seen to vary over a wide range because of the variable behavior of the preceding biological nitrification system. Actual influent concentrations varied between 35-130 mg/l. With hydrated lime doses as low as 50 mg/l the BOD levels in the clarifier effluent were consistently reduced to 5 mg/l or less. The effective BOD removals at relatively low lime doses show a close correlation to the preceding curve regarding the behavior of suspended solids, and indicate that nearly all of the BOD discharged from the biological nitrification system is in the suspended form. At lime doses above 50 mg/l the BOD in the filter effluent was consistently below 5 mg/l, in spite of the fact that the BOD in the influent to the clarification system was highly variable.

Lime precipitation of those constituents contributing to COD was not as complete as in the case of BOD. This is shown in Figure 18 which relates influent and effluent COD levels to the applied lime dose. During the period of this study, the COD concentrations influent to the lime system varied from 25 through 175 mg/l. As with the BOD concentration, a significant reduction in COD was noted at lime doses of less than 100 mg/l. However, for COD the observed reduction amounted to about 65-75% as compared to 90-95% for BOD. This difference in effectiveness reflects the fact that a substantial fraction of the COD in nitrified secondary effluent is contributed by soluble organic species. Additional treatment, such as carbon adsorption, is indicated for removing this fraction of the COD.

Phosphorus analyses were performed to identify both the ortho- and total phosphate concentration variations through the system. The orthophosphate concentrations in the nitrification clarifier effluent varied over the period of study from 8 mg/l $PO_4$ to 32 mg/l $PO_4$. A corresponding variation in total influent phosphorus was observed in the concentration range of 10-39 mg/l total phosphate. As shown in Figure 19, the orthophosphate concentration was reduced from the average influent of 22 mg/l $PO_4$ to an effluent value of 4.5-5 mg/l $PO_4$ at a lime dose of 100 mg/l,

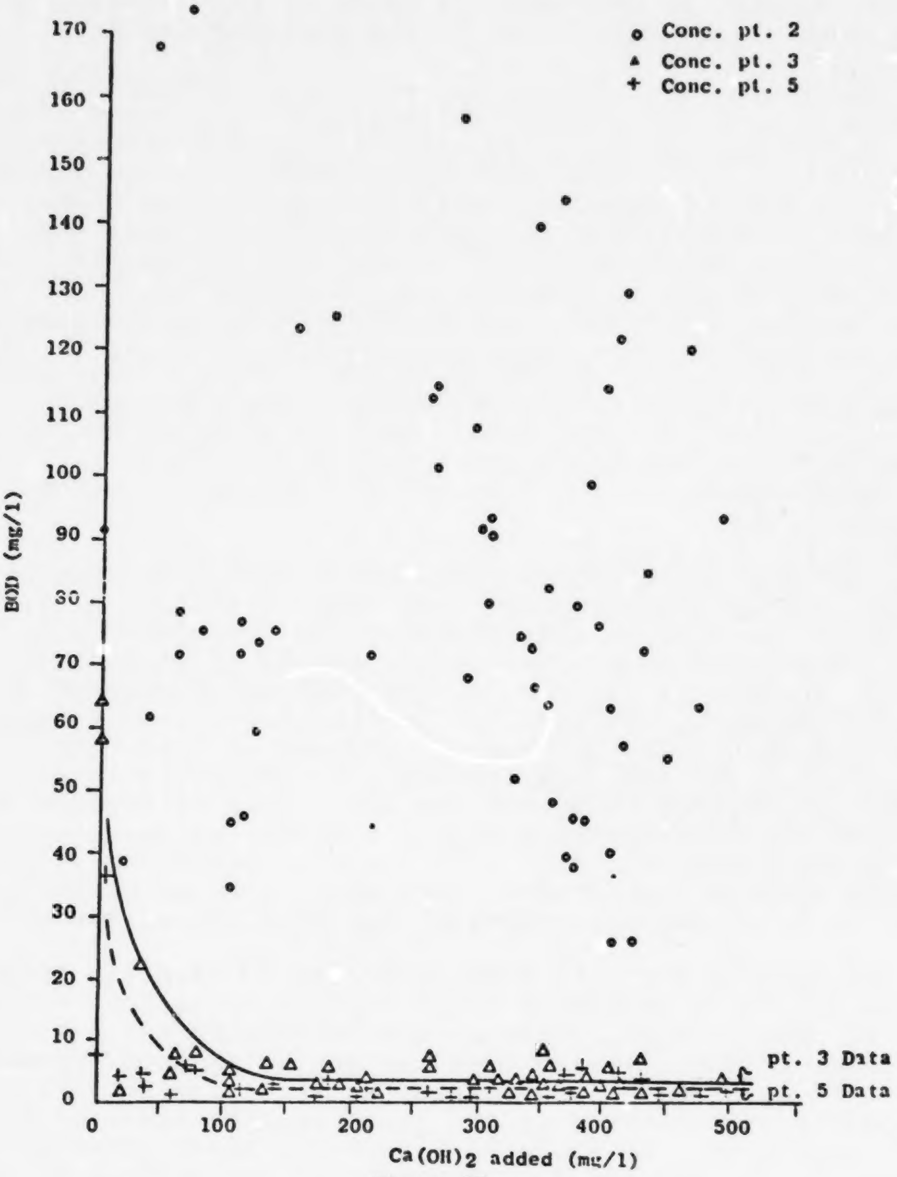

Figure 17

BOD REMOVAL WITH LIME ADDITIONS

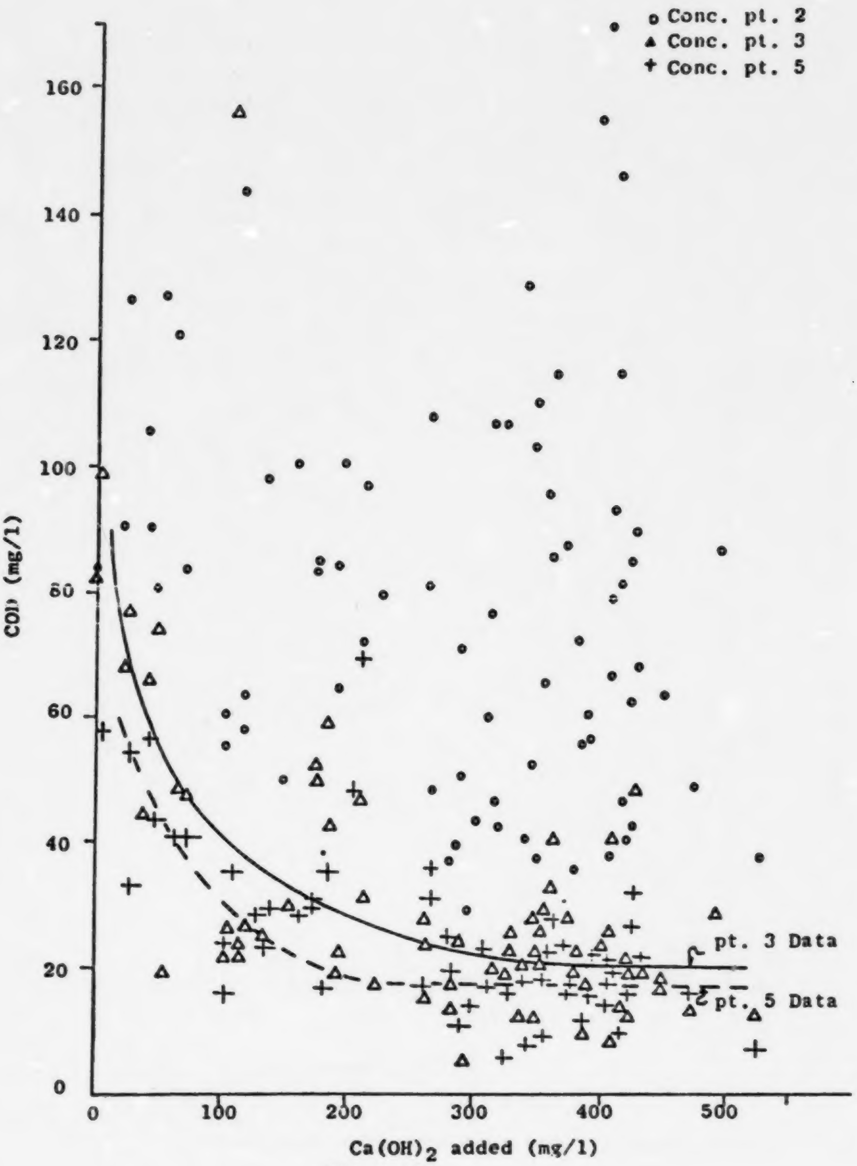

Figure 18

COD REMOVAL WITH LIME ADDITION

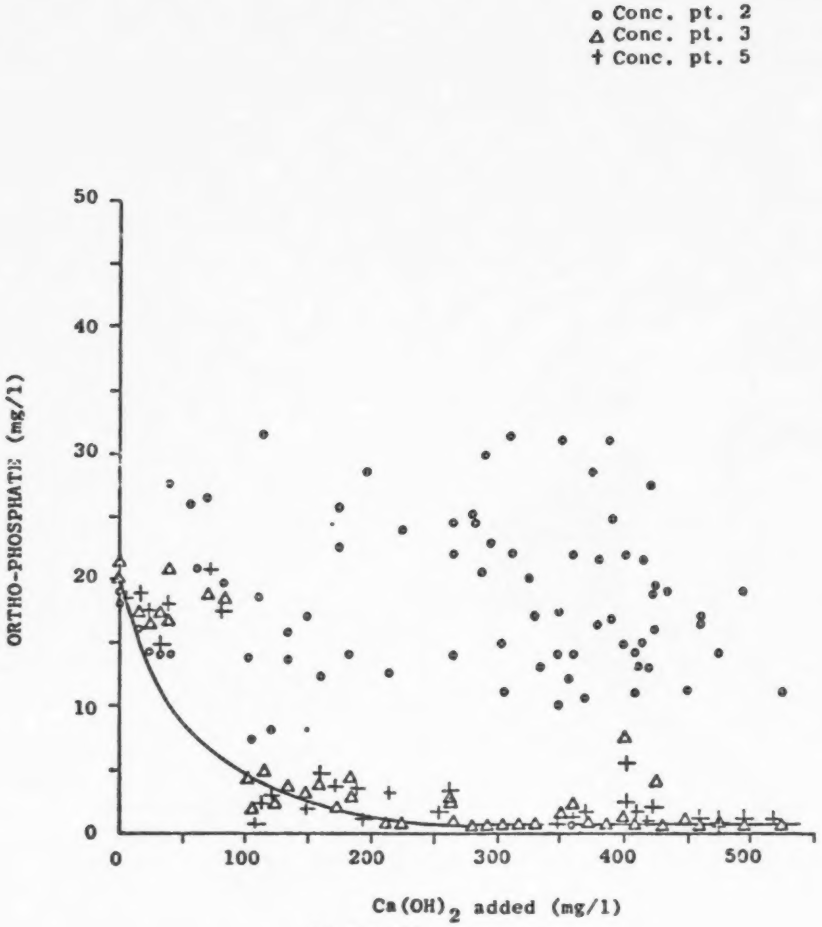

**Figure 19**

ORTHO-PHOSPHATE REMOVALS WITH LIME ADDITIONS and to a lower value of about 1 mg/l $PO_4$ for lime doses of 300 mg/l and greater. Likewise, for total phosphates, an 80% reduction, from an average of 25 mg/l $PO_4$ to about 5 mg/l $PO_4$, was observed with a lime addition of 100 mg/l to the system. Above lime doses of 300 mg/l the residual total phosphate concentration was consistently below the level of 1 mg/l $PO_4$. The close correlation between the phosphorus levels after settling and filtration is likely associated with the highly efficient sludge blanket clarification which was designed into the settling units of the pilot plant. As indicated by Jenkins (3), it would be expected that in a plant scale clarifier, sufficient phosphate precipitate would carry-over to necessitate subsequent filtration for consistent high level phosphorus removals.

The 80% reduction of total phosphate at the 100 mg/l lime dose, corresponding to a pH of about 9.5, confirms on pilot scale the appreciable removals predicted by Schmid and McKinney (27), in a somewhat different treatment context. Removals at this level may be significant in the sense of upgrading existing wastewater treatment plants, since substantial phosphorus is removed at this dose together with essentially all of the suspended particulates and BOD.

The efficiencies of removal for the major wastewater constituents are shown graphically in Figures 21 through 25. From these curves it is evident that the most significant percentage removals are realized for all the constituents at a lime dose of 100 mg/l or less. Doses above this level serve to minimize the absolute quantities of residual pollutants.

In an effort to reduce the lime dose requirement for effecting high level phosphate removals, a sludge recycle mode was established to provide nucleation sites for improving the efficiency of lime utilization. In this operational mode, variable fractions of the chemical sludge blowdown were recycled back to the system ahead of the rapid mix unit. For the most effective case, with 50% recycle in the sludge system, the phosphorus removals are compared to the conventional operating mode in Figure 26. While the recycle removals at low lime dose are consistently on the low side of the residual phosphate envelope, the pilot plant data do not appear to show a highly significant deviation from the results developed in the conventional mode of operation at the phosphorus removal levels that are generally considered desirable. Thus, on a plant scale, it is questionable if the recycle mode is justified.

Throughout the entire lime clarification study, weekly samples were taken at various points in the conventional clarification system for analysis of the concentration of coliforms, fecal coliforms, and fecal streptococcus. The data from these

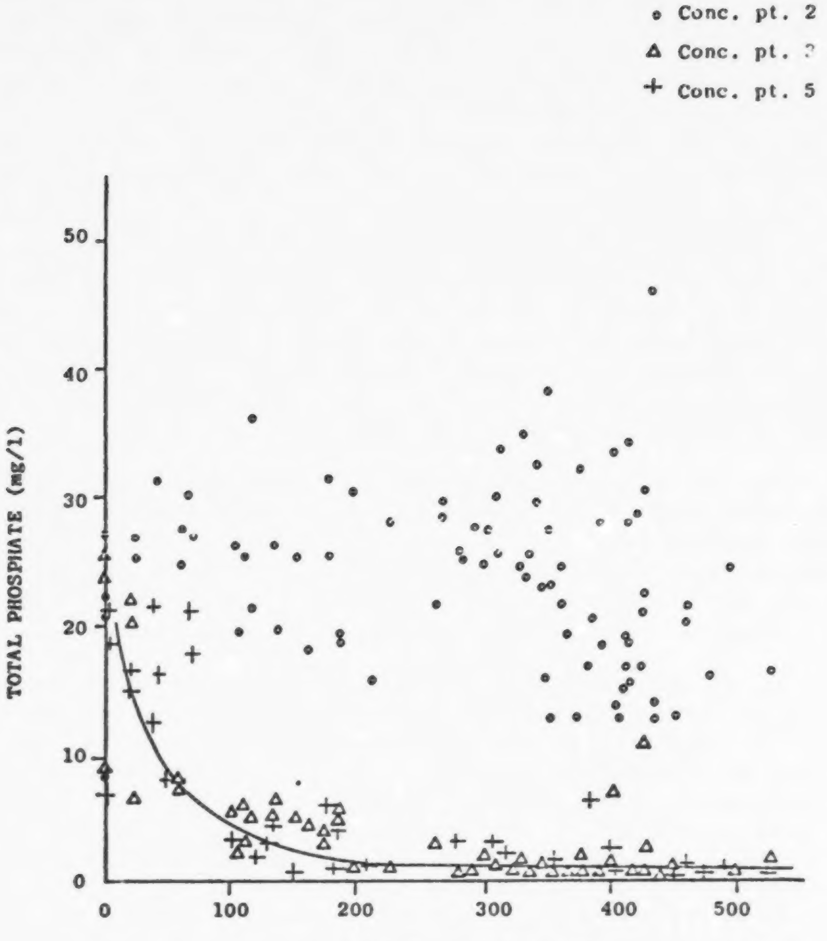

Figure 20

TOTAL PHOSPHATE REMOVAL WITH LIME ADDITIONS

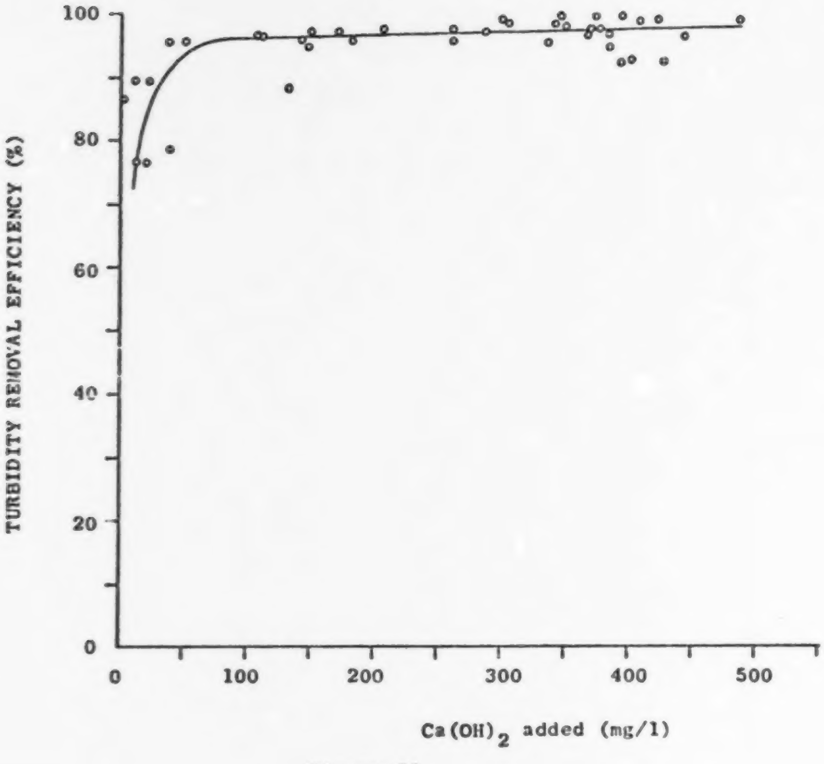

Figure 21

TURBIDITY REMOVAL EFFICIENCY AFTER LIME TREATMENT AND FILTRATION

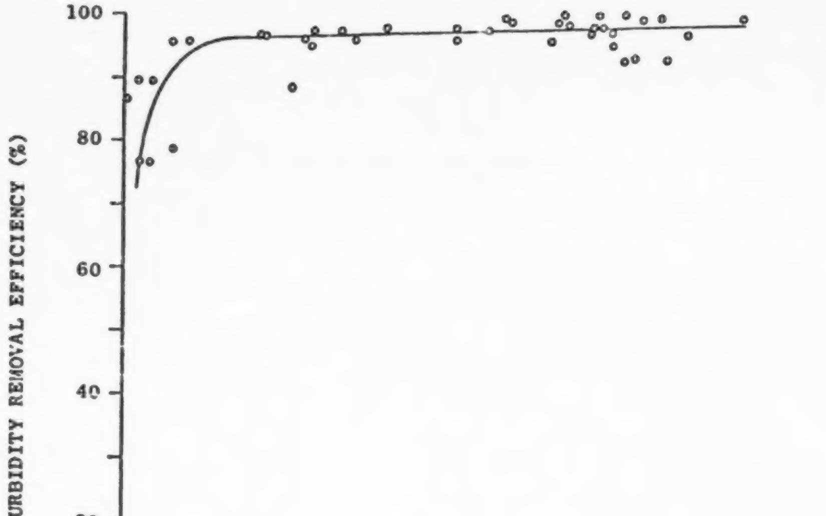

Figure 21

TURBIDITY REMOVAL EFFICIENCY AFTER LIME TREATMENT AND FILTRATION

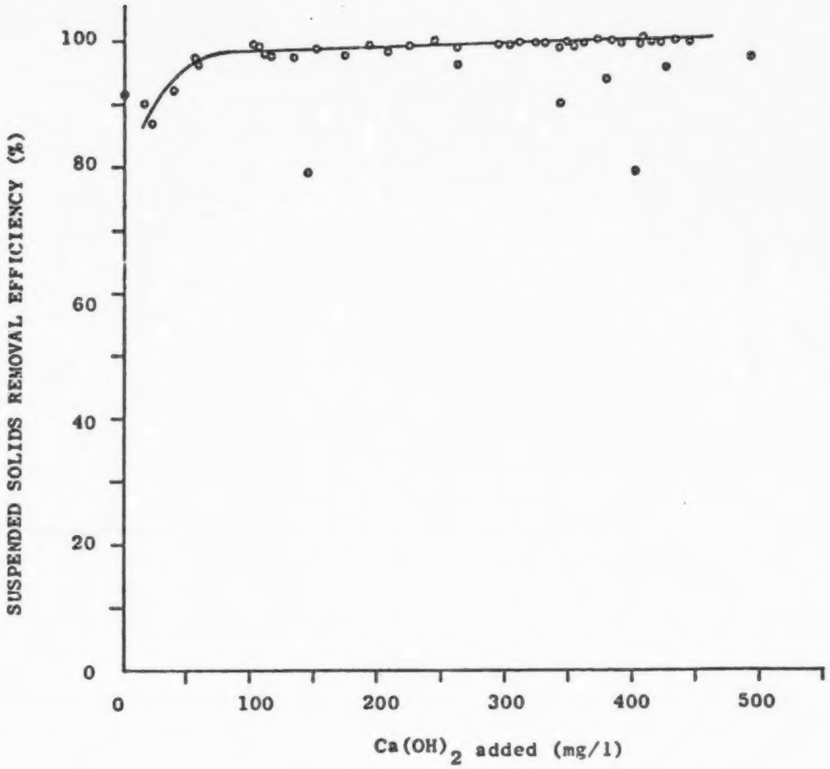

Figure 22

SUSPENDED SOLIDS REMOVAL EFFICIENCY AFTER LIME TREATMENT AND FILTRATION

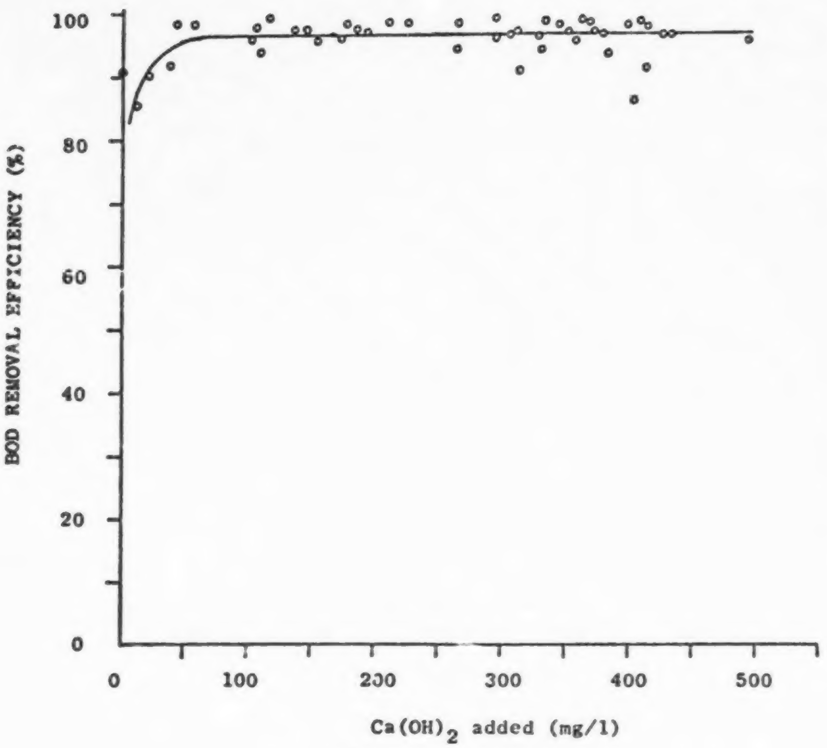

Figure 23

BOD REMOVAL EFFICIENCY AFTER LIME TREATMENT AND FILTRATION

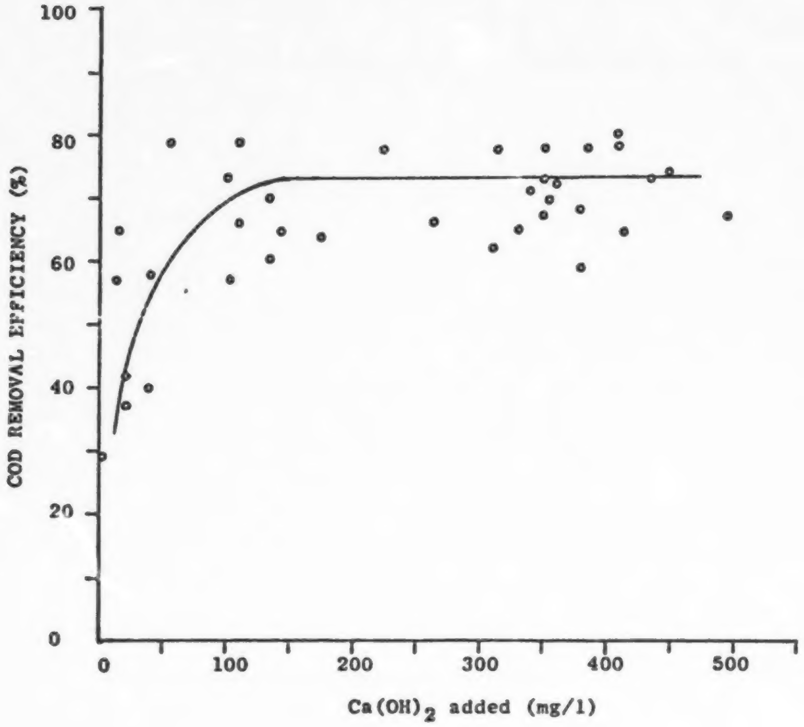

Figure 24

COD REMOVAL EFFICIENCY AFTER LIME TREATMENT AND FILTRATION

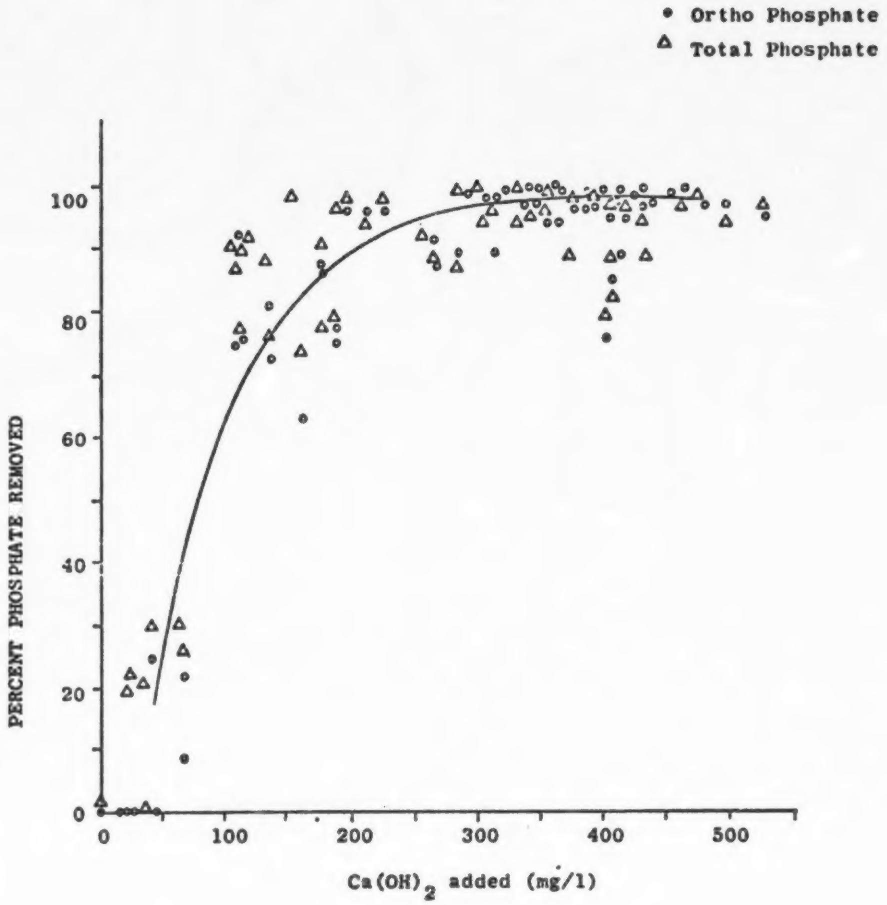

Figure 25
PHOSPHATE REMOVAL EFFICIENCY WITH LIME

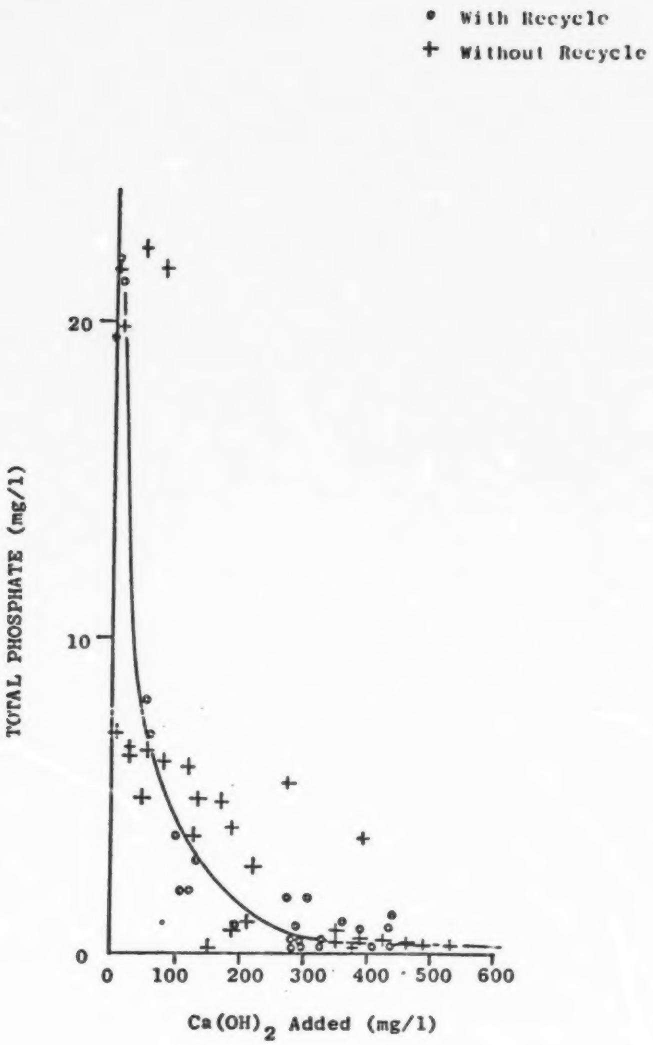

Figure 26

EFFECT OF LIME SLUDGE RECYCLE ON THE REMOVAL OF TOTAL PHOSPHATE WITH LIME

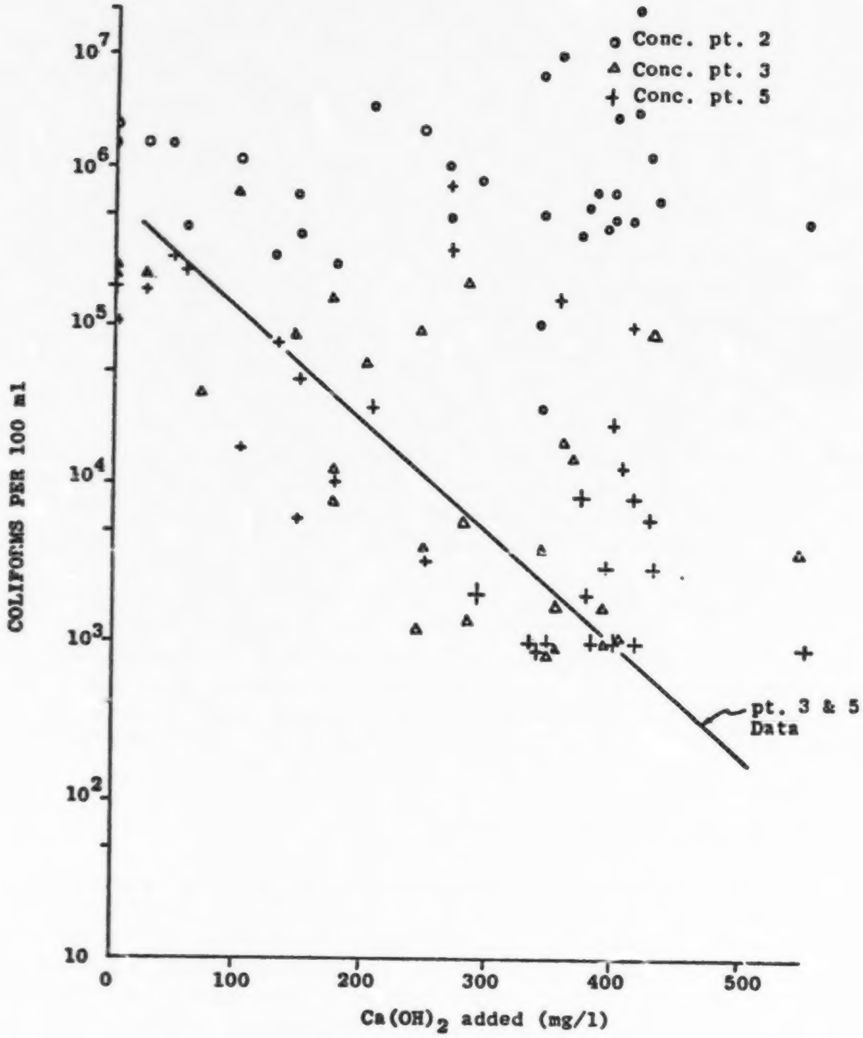

Figure 27

COLIFORM REMOVAL WITH LIME ADDITIONS

analyses have been reviewed with respect to the applied lime dose, as shown in Figure 27 for the total coliform concentration. While there is considerable scatter in the data, there does appear to be a definite proportional tendency between the applied lime dose and the removal of coliform organisms. This is consistent with the laboratory observations of several authors who have noted that the degree of effectiveness of lime in destroying bacterial organisms is a function of the pH developed within the system (31)(32)(33). At lime doses of about 400 mg/l, corresponding to pH values of approximately 11.5, there was a greater than two-fold reduction in coliform removals within the system. Similar trends were observed for the fecal coliforms and fecal streptococcus as shown in Figures 28 and 29.

Trace metal analyses were conducted during the study period for iron, copper, manganese, chromium, lead, zinc, cadmium, and molybdenum by the quality control laboratory of the Denver Water Department. In these studies, all analyses were made by atomic adsorption methods as discussed in the 12th Edition of Standard Methods.

The data are summarized in Figures 30 through 38, where the temporal variations of both influent and effluent concentrations are plotted for each of the elements. From the average influent and effluent concentrations presented in Table VI, it can be seen that the concentrations of Zn, Cr, Mn, Cu, Cd, and Fe were reduced somewhat by the lime treatment, while the concentrations of Al, Mo, and Pb were not significantly affected at the levels encountered in the secondary effluent investigated in these studies. Above a lime dose of about 100 mg/l there did not appear to be any significant dose dependence on the degree of removal for the elements which were investigated.

## Sludge Production and Thickening

One of the important considerations in the successful utilization of lime for tertiary treatment of wastewater is the quantity and character of sludge requiring disposal. To characterize the sludge generated in this study, the solids collected in the clarifier were blown down and analyzed for volume, solids concentration, and thickening behavior. From these analyses, data were developed which yielded information on the quantity of sludge requiring disposal for a given lime dose. This data is summarized in Figure 39. From this curve, it can be seen that there is a linear relationship between the applied lime dose and the quantity of sludge requiring disposal. The slope of this curve for the wastewater treated in these studies was 0.56 tons/M.G./100 mg $Ca(OH)_2$ added per liter. Of this quantity the volatile solids comprised approximately 10-15%.

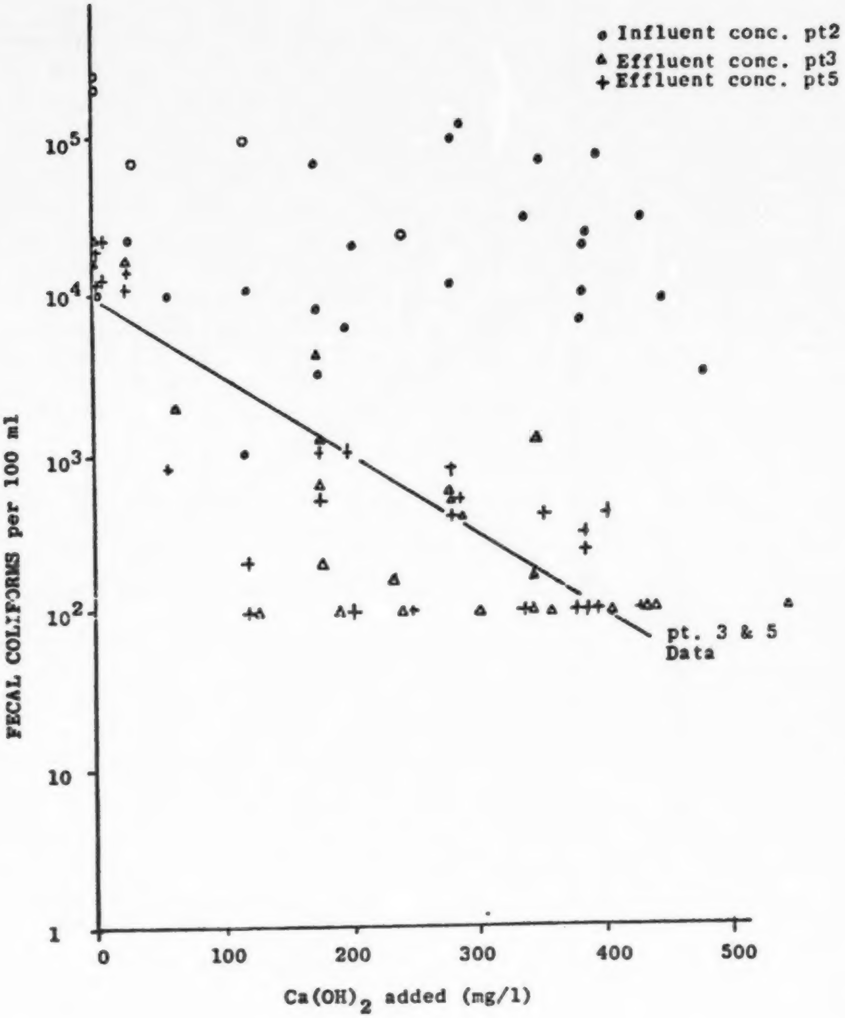

Figure 28

FECAL COLIFORM REMOVALS WITH LIME ADDITIONS

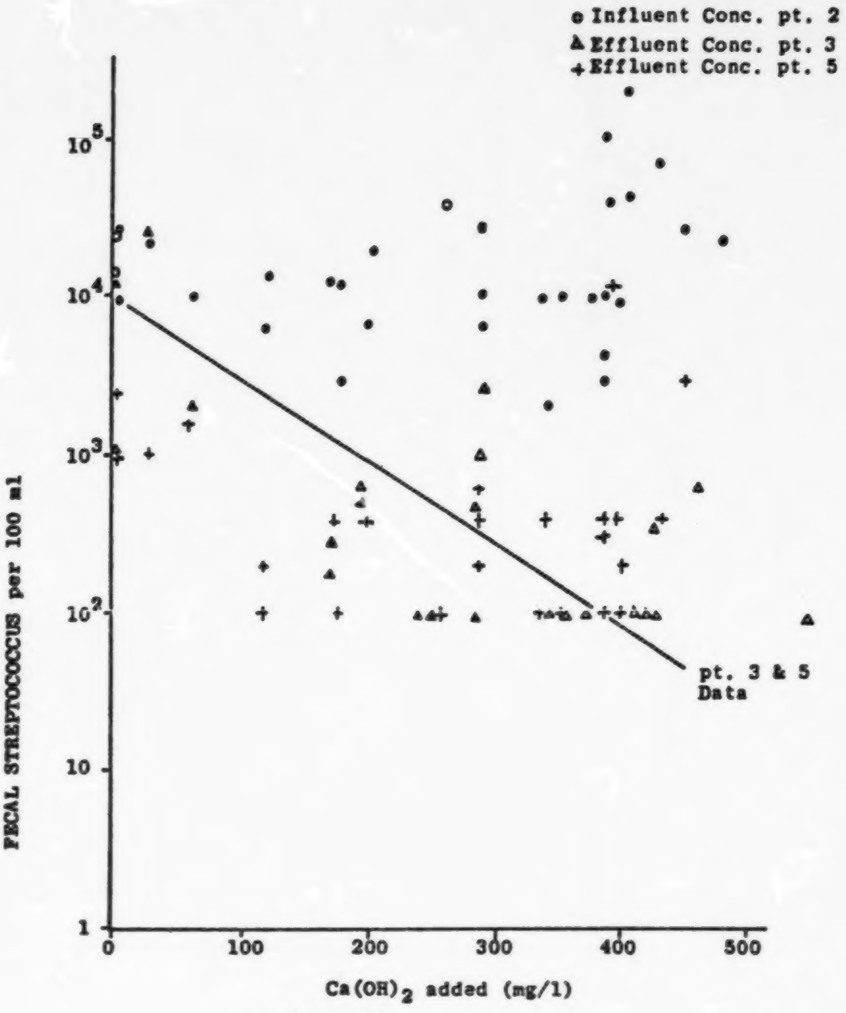

**Figure 29**

FECAL STREPTOCOCCUS REMOVALS WITH LIME ADDITION

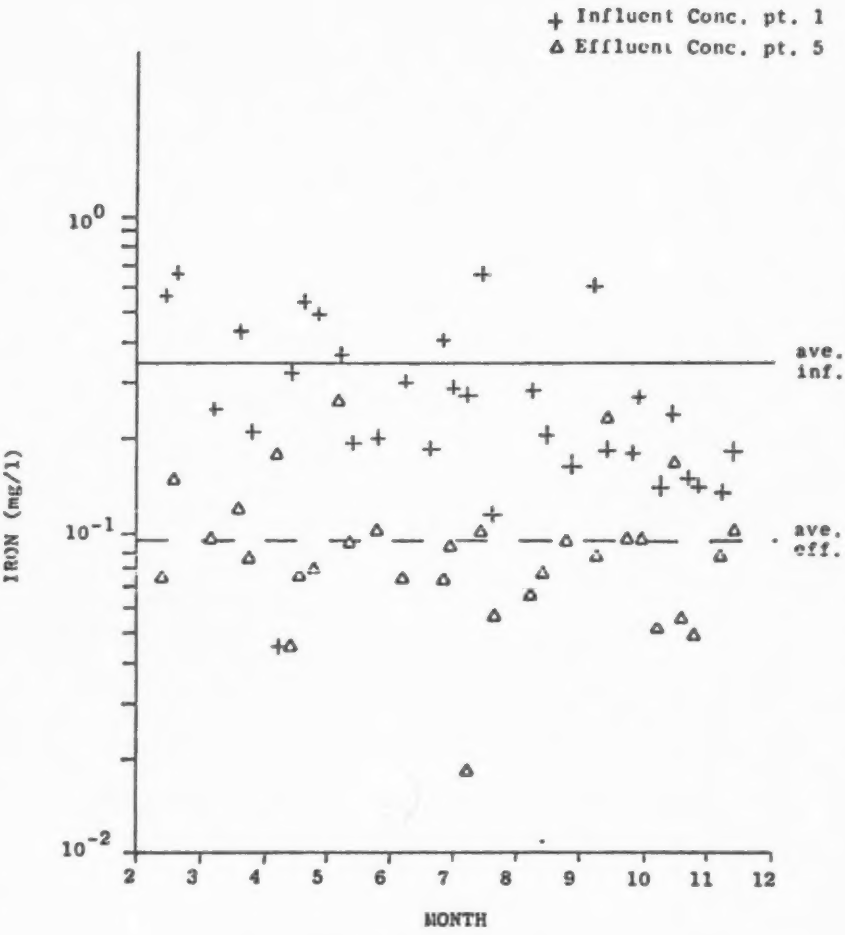

Figure 30

MONTHLY VARIATIONS IN IRON CONCENTRATION

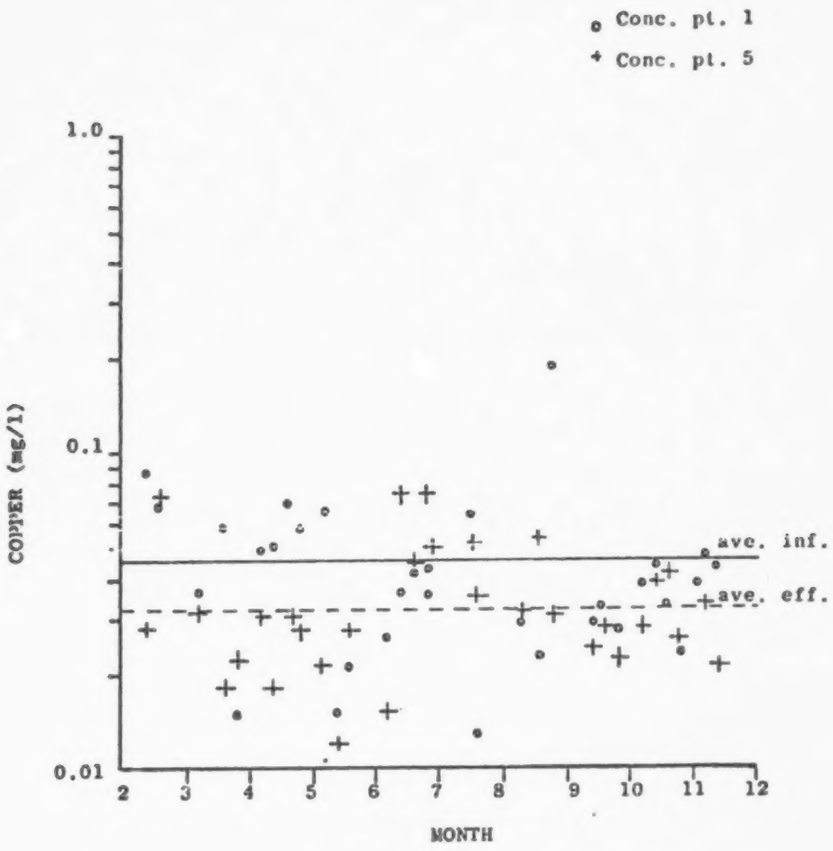

Figure 31

MONTHLY VARIATIONS IN COPPER CONCENTRATION

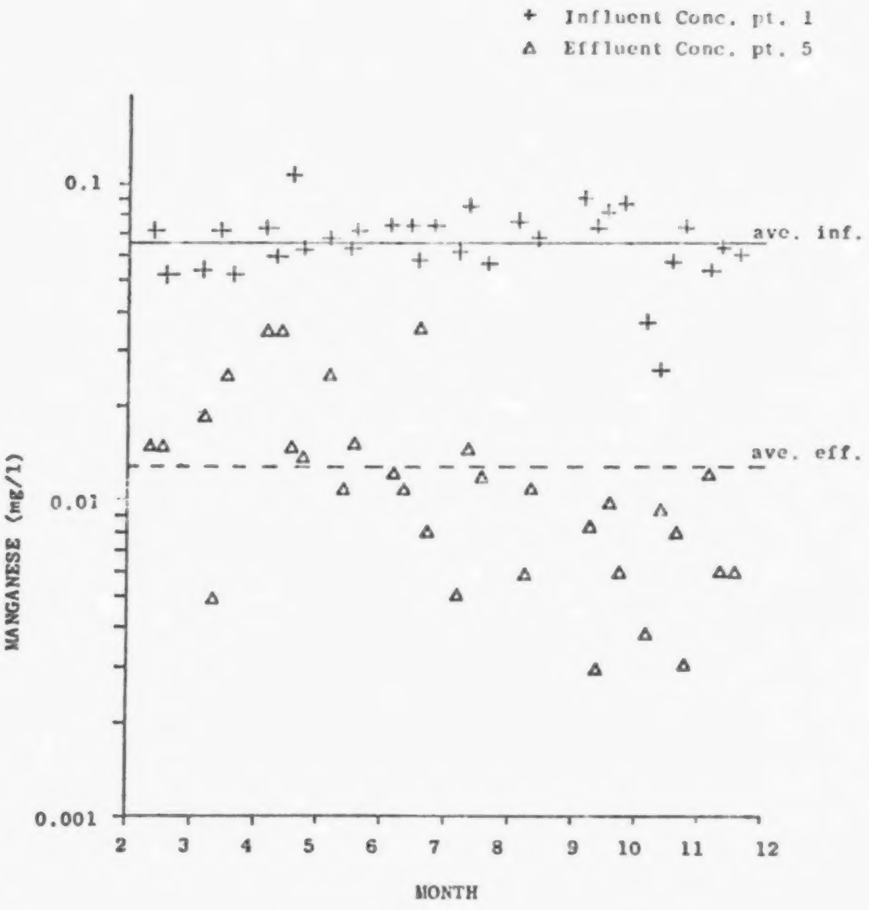

Figure 32
MONTHLY VARIATIONS IN MANGANESE CONCENTRATION

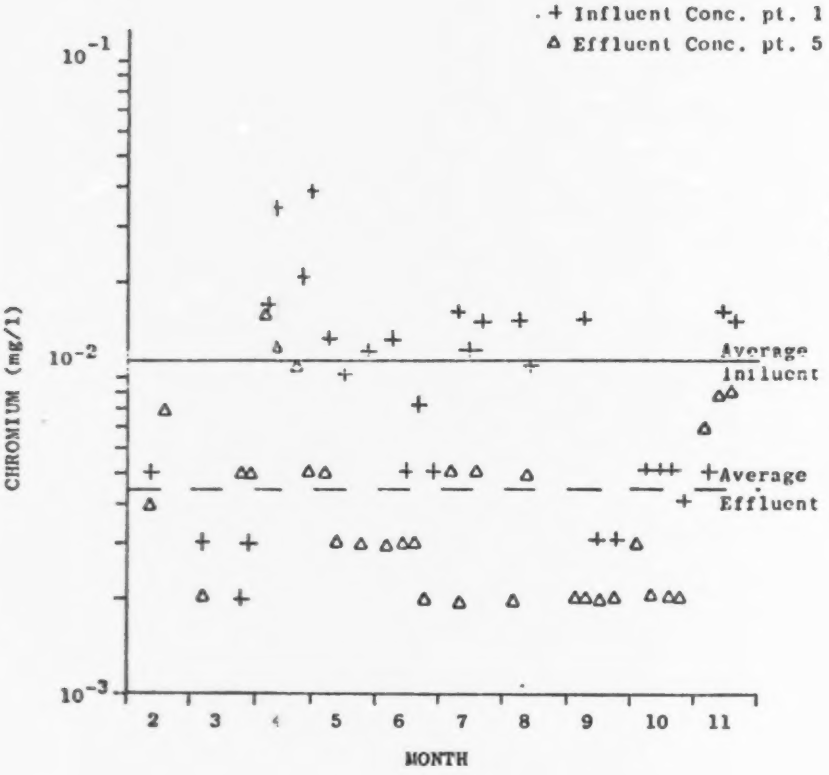

Figure 33

MONTHLY VARIATION IN CHROMIUM CONCENTRATION

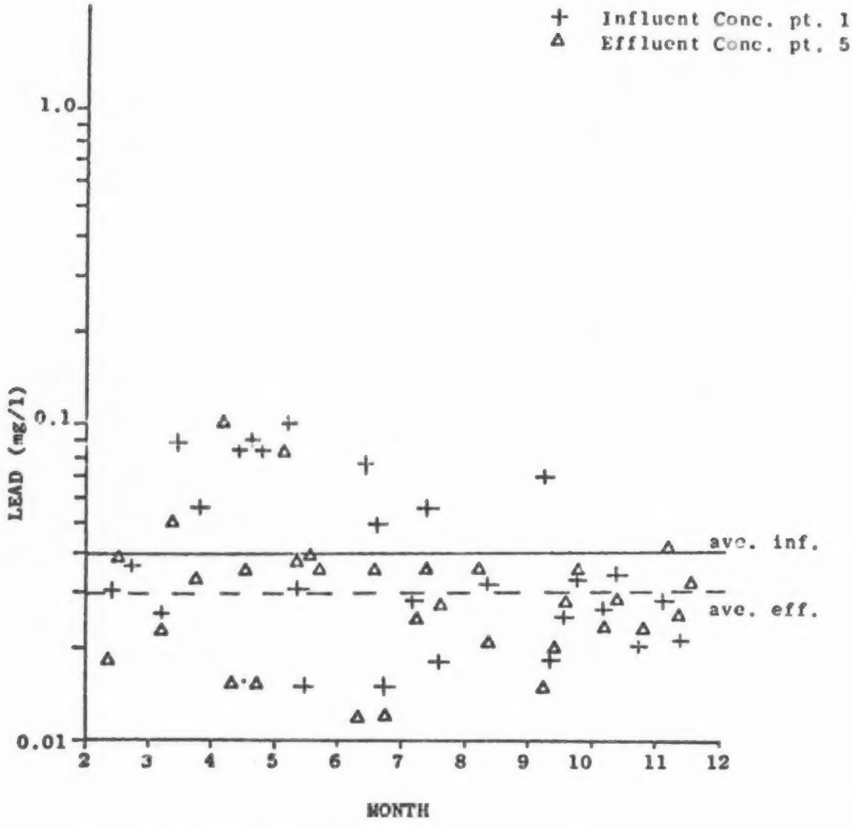

**Figure 34**

MONTHLY VARIATIONS IN LEAD CONCENTRATION

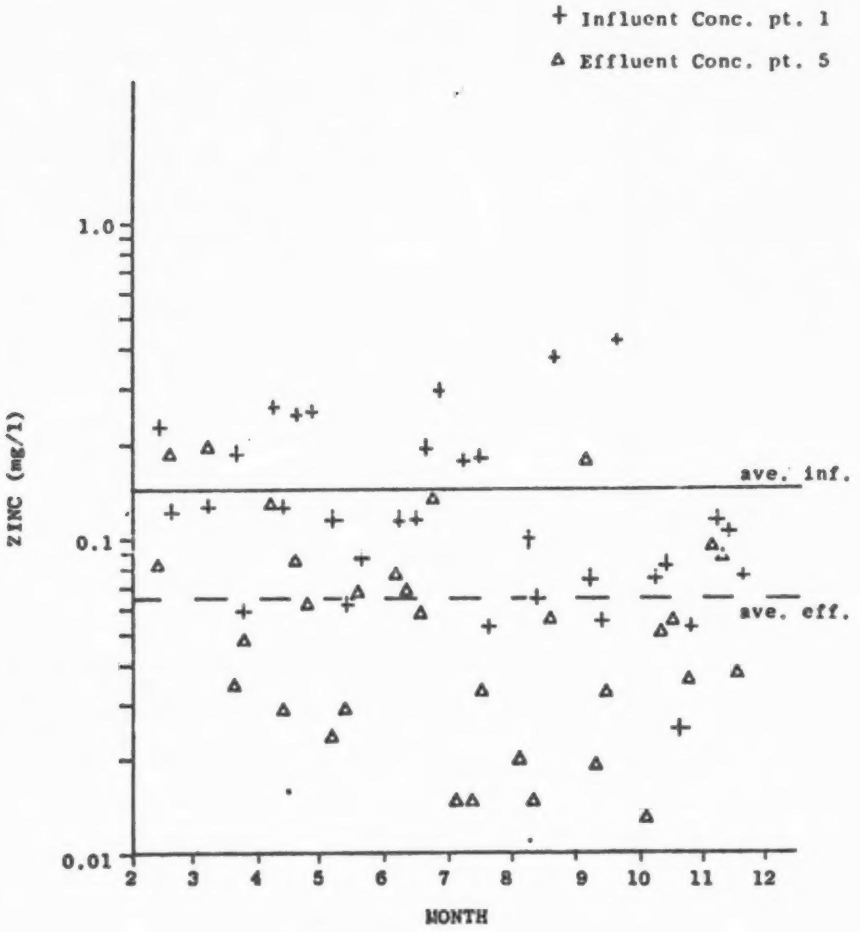

**Figure 35**

MONTHLY VARIATIONS IN ZINC CONCENTRATIONS

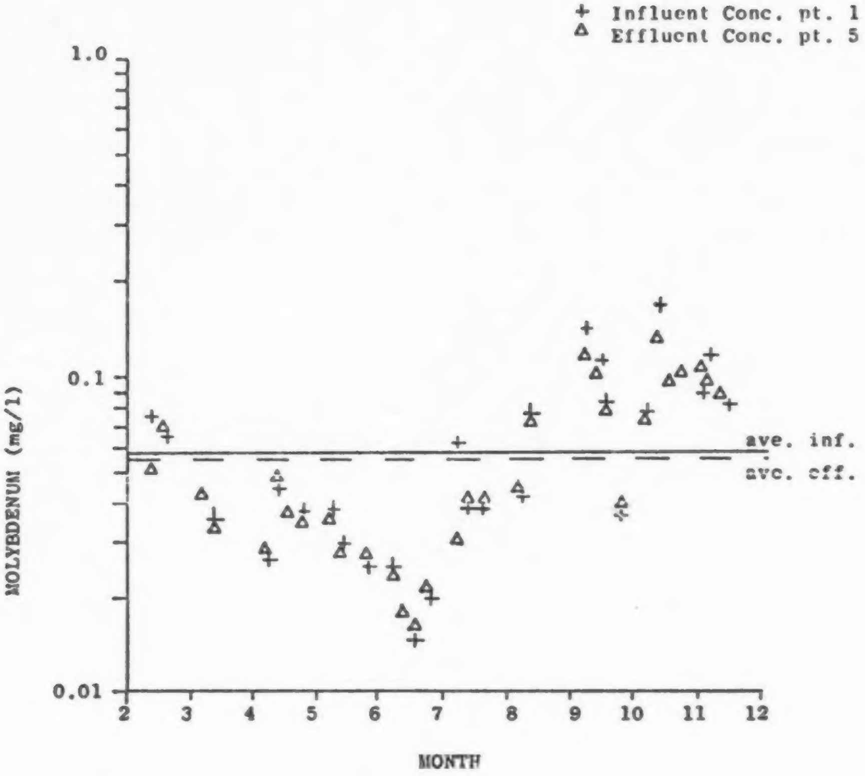

Figure 36

MONTHLY VARIATION IN MOLYBDENUM CONCENTRATION

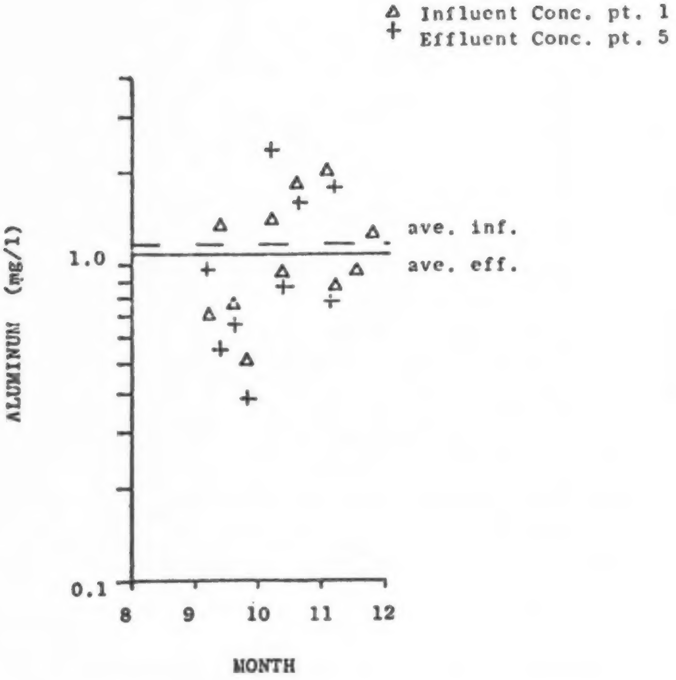

Figure 37

MONTHLY VARIATION IN ALUMINUM ION CONCENTRATION

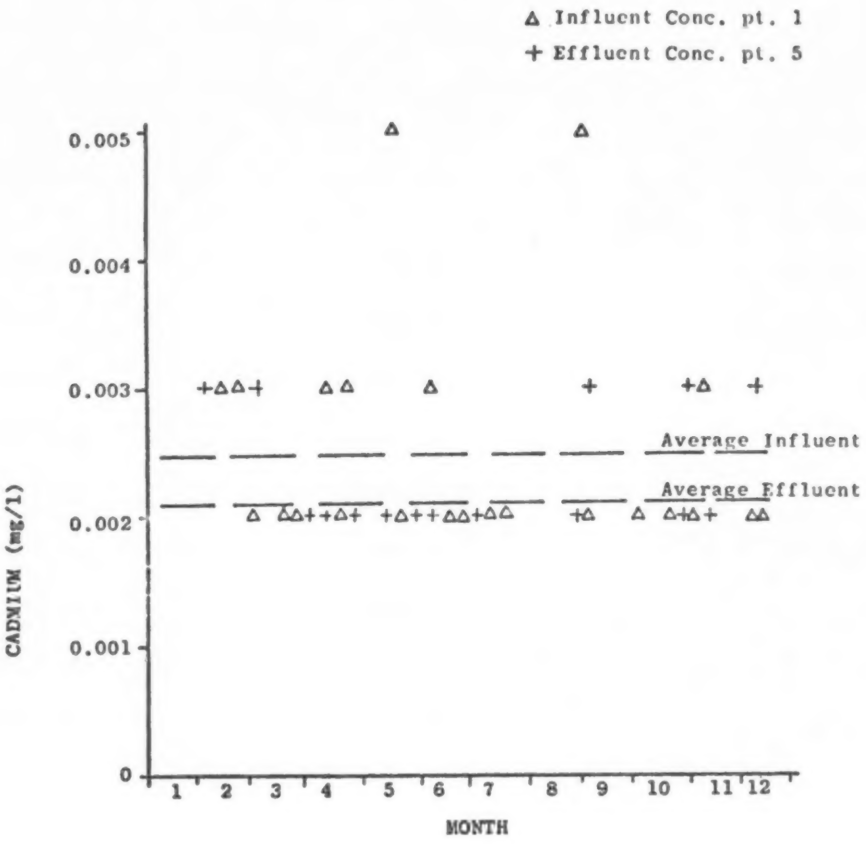

Figure 38

MONTHLY VARIATIONS IN CADMIUM CONCENTRATION

## Table VI

## Trace Metal Removals

| Metal | Average Influent Concentration Point 1 (mg/liter) | Average Effluent Concentration Point 5 (mg/liter) |
|---|---|---|
| Aluminum | 1.110 | 1.120 |
| Molybdenum | 0.057 | 0.055 |
| Zinc | 0.141 | 0.065 |
| Lead | 0.040 | 0.030 |
| Chromium | 0.0115 | 0.0045 |
| Manganese | 0.065 | 0.013 |
| Copper | 0.046 | 0.032 |
| Iron | 0.356 | 0.097 |
| Cadmium | 0.0025 | 0.0021 |

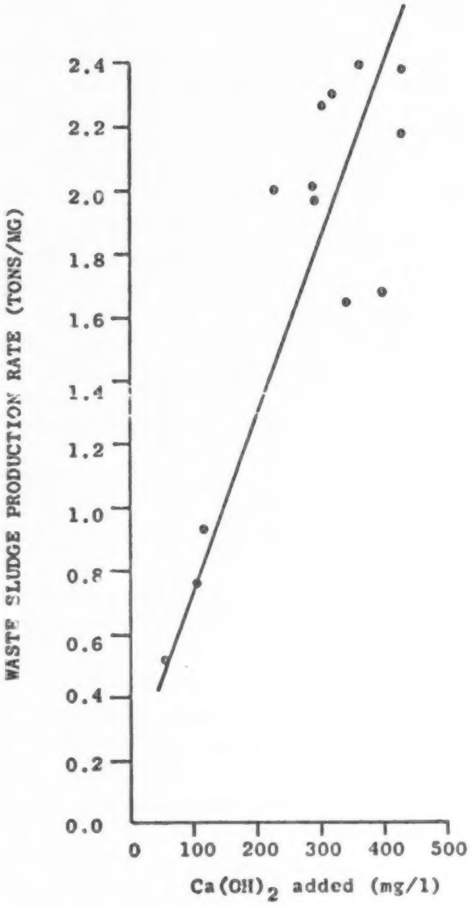

Figure 39

WASTE SLUDGE PRODUCTION (TONS/MG) AS A FUNCTION OF LIME DOSE

Solids concentrations in the sludge blowdown were typically between 1 and 4%. These concentrations could be approximately doubled with two hours of quiescent settling. This concentration would be expected to increase further in a gravity thickening device which incorporated some form of mixing.

## Filter Headloss

Since the rate of development of filter headloss is very important to the operation and economics of filtration, several extended filtration runs were made on the lime treated clarifier effluent. These runs provided an indication of the practical length of filter runs at each of several loading rates. The data from these experiments are plotted in Figure 40 with filter headloss as a function of the grams of solids removed on the filter per cubic foot of filter bed depth. The abscissa unit was chosen to normalize for the influence of variable influent suspended solids. As expected, the rate of headloss development increased as the hydraulic loading on the filter was increased from 1.4 to 2.1 gpm/ft$^2$. At the 2.1 gpm/ft$^2$ loading rate, the filter headloss reached 8 ft. at a total solids loading of about 13 grams per cubic foot. For typical suspended solids levels of 5 mg/l in the clarifier overflow, this represents a filter run of about 16 hours, assuming that all of these solids are removed on the filter. This definitely represents a practical operating range.

## Alum Dose Variation

For purposes of comparing the effectiveness of alum and lime in the conventional clarification system, a series of studies was performed utilizing alum as the chemical additive. In these studies the slow mix detention time was maintained at 48 minutes, the flocculation intensity was constant at 19 rpm, and the pH was maintained in the range 5.5-6. The alum dose was varied, together with the filter loading rates, and determinations were made to define the extent of removal for turbidity, suspended solids, BOD, COD, and the phosphate forms. Additionally, the effect of variable dosing and filter loading rates was analyzed for the impact on the rate of development of filter headloss.

The effectiveness of tertiary alum treatment in clarifying nitrified secondary effluent is graphically demonstrated in Figure 41. With alum addition followed by settling, the wastewater turbidity was reduced from an influent value of 13.5 mg/l

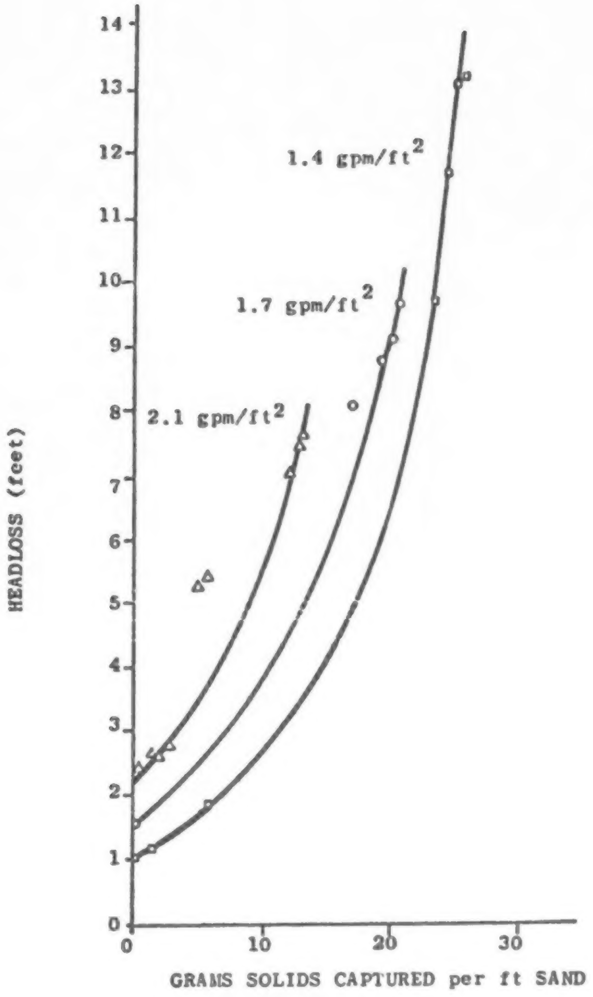

Figure 40

HEADLOSS AS A FUNCTION OF SOLIDS CAPTURE IN A RAPID SAND FILTER

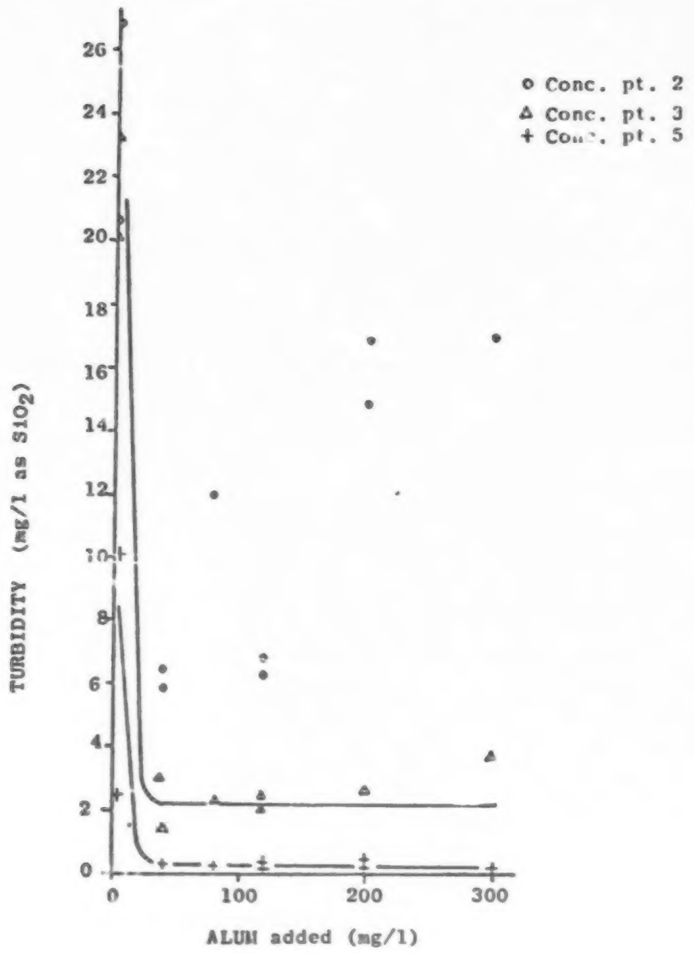

Figure 41

TURBIDITY REMOVAL WITH ALUM IN THE ALUM CLARIFIER SYSTEM

$SiO_2$ to about 2.5 mg/l $SiO_2$ at an alum dose of only 50 mg/l. With subsequent rapid sand filtration the turbidity was reduced further to less than 1 mg/l. At higher alum doses, up to 300 mg/l, the filter effluent showed an additional slight improvement. However, at this level the filter experienced increased solids loadings as indicated by the increase in turbidity from the sedimentation unit. Thus, operation at these high alum levels for turbidity removal would adversely affect the length of filter runs.

Accompanying the removal of suspended particles, alum provided a significant reduction in the wastewater BOD. At alum doses as low as 50 mg/l, the filter effluent BOD was reduced to less than 2 mg/l. This level is consistent with the BOD concentration attainable utilizing lime in the conventional clarification system, and confirms that the majority of BOD contributing organics in nitrified secondary effluent are in the suspended form. Likewise, the removal of COD followed the pattern established in lime treatment. It was possible to reduce the filter effluent COD to about 20 mg/l at an alum dose of approximately 50 mg/l. Higher alum doses did not significantly improve the removal of this constituent. Thus, as shown in the lime clarification data, chemical treatment without carbon adsorption was limited to approximately 65-75% COD reductions for the wastewater treated in this investigation.

Alum was also effective in reducing the concentration of phosphate in wastewater. This is shown in Figures 44, and 45, respectively. Approximately 80% of the ortho- and total phosphate species were precipitated from the clarifier effluent at an alum dose of 150 mg/l. Filtration of this settled product water reduced the average phosphate concentrations to less than 1 mg/l. At alum doses above 150 mg/l, there was little additional improvement in the degree of phosphorus removal. These alum dose requirements are in close agreement with the data developed in the pilot plant experiences at the Bay Park Sewage Treatment Plant in Nassau County, New York (50,51). Likewise, the phosphorus levels attainable in the product water are of the same order of magnitude. The importance of filtration in realizing the full potential of phosphate precipitation is clearly indicated by the results of this study. For the experimental system used in this investigation it was difficult to control some discharge of phosphate solids in the clarifier overflow.

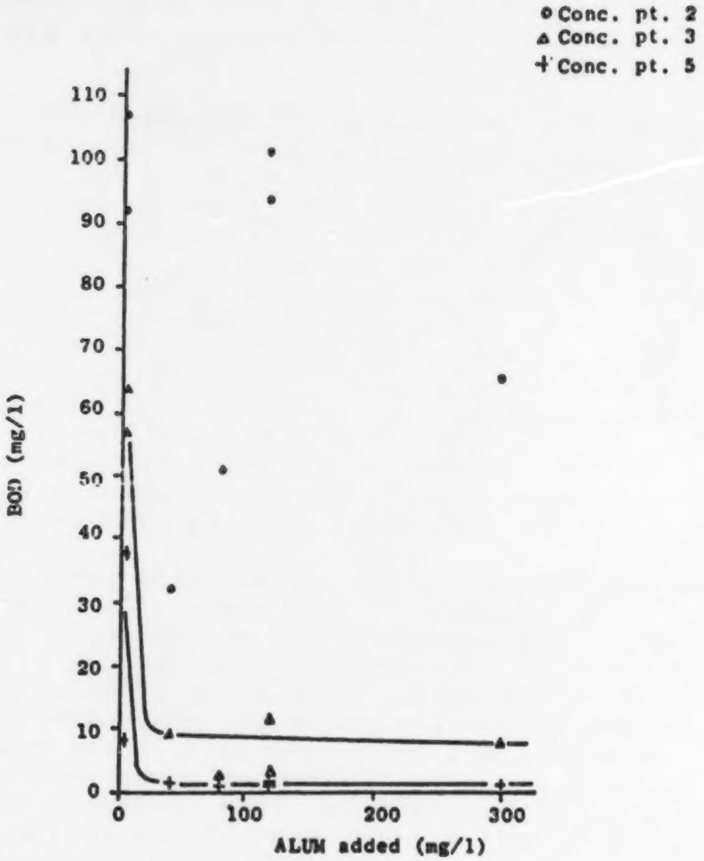

**Figure 12**

BOD REMOVAL WITH ALUM IN THE ALUM CLARIFIER SYSTEM

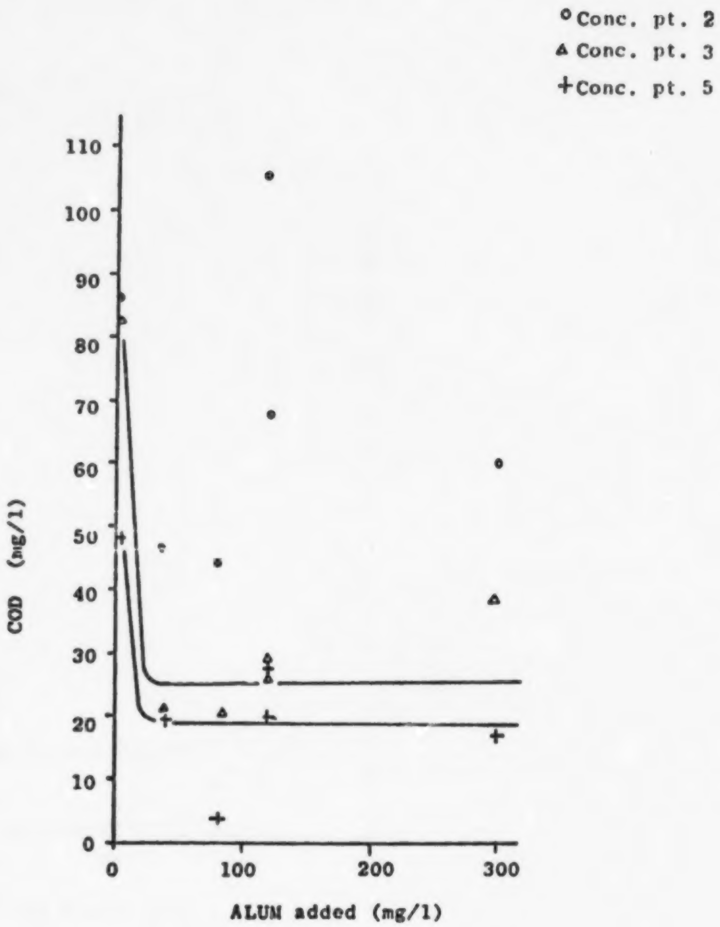

**Figure 43**

COD REMOVAL WITH ALUM IN THE ALUM CLARIFIER SYSTEM

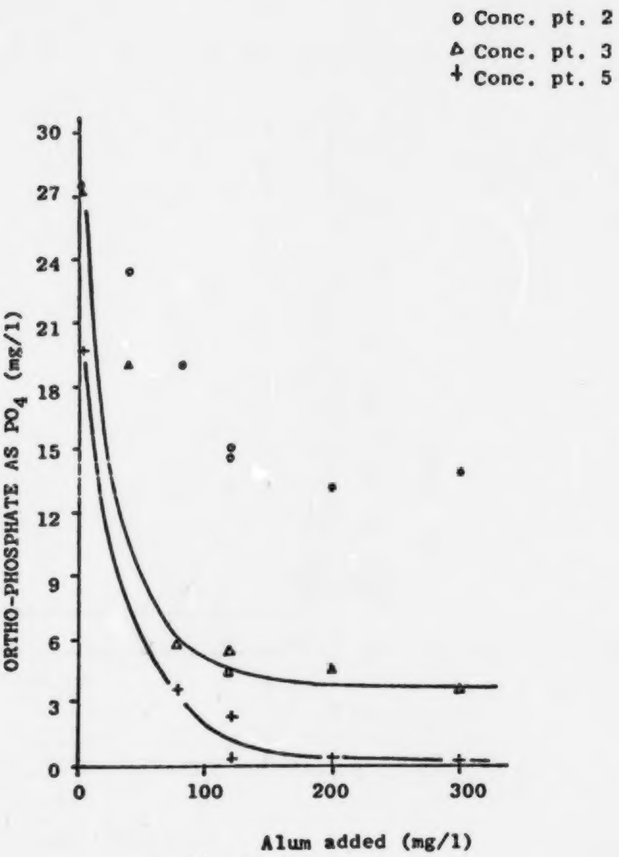

Figure 44

ORTHO-PHOSPHATE REMOVAL WITH ALUM IN THE ALUM CLARIFIER SYSTEM

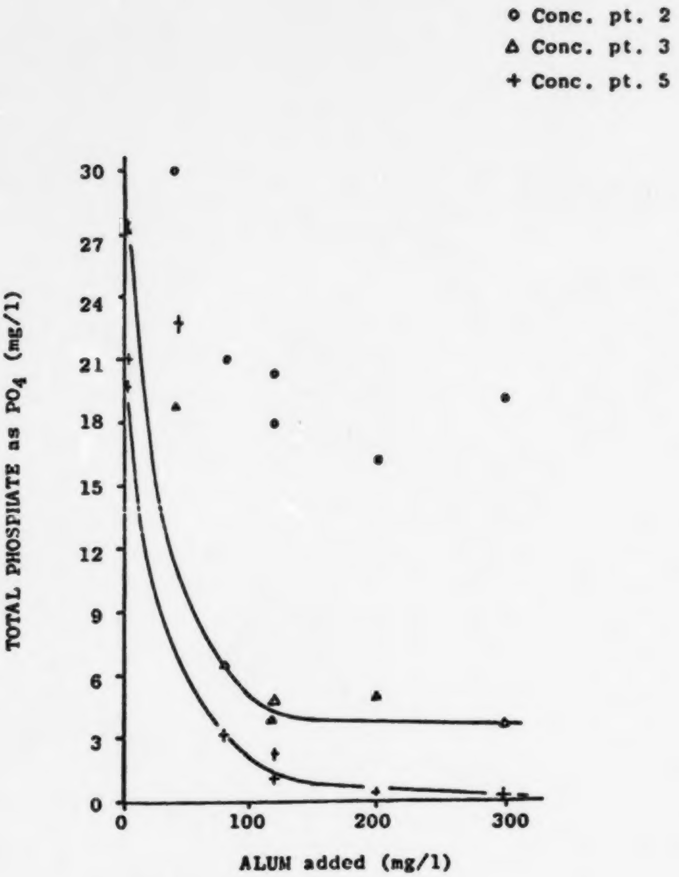

Figure 45

TOTAL PHOSPHATE REMOVAL WITH ALUM IN THE ALUM CLARIFIER SYSTEM

# VIII. HIGH RATE CLARIFICATION

## INTRODUCTION

An alternative system of advanced wastewater clarification treatment is the direct dosing of chemically-treated wastewater onto dual-media filters. Unlike the conventional AWT process lineup, this alternative system eliminates the sedimentation step and provides for the coagulated wastewater to be processed directly onto the dual-media filter (36). Most of the research which has been conducted on this type of system has been in the treatment of potable water supplies. Conley and Hsiung (37) have reported on experiences with some of the numerous water treatment plants employing direct multi-media filtration of coagulated water. They report one case with 12-hour filter runs at a hydraulic loading of 5 gpm/ft$^2$ with chemical addition of 32 mg/l alum, and 0.2 mg/l polymer. In this system 25 units of turbidity were applied directly to the filter without prior flocculation or settling. Filtered water turbidities ranged from 0.2 to 0.4 Jackson Turbidity Units.

The concept of using direct, dual-media filtration for clarification of chemically coagulated secondary wastewater effluents is a fairly new idea. The operation appears to have potential for the following reasons: 1) Significant cost and space savings may result from elimination of the chemical clarifier, 2) Continued development of a more ideal coarse-to-fine filter media will allow higher filter efficiencies, 3) Constantly improving automated backwash equipment can easily handle the necessary chore of more frequent backwash operations under the higher filter solids loading conditions inherent in direct filtration, and 4) The process is a versatile one--if only solids removal is desired, low chemical doses can be used to coagulate the solids which can then be removed from the liquid stream on the mixed-media filter. If additional treatment is desired, such as for phosphate removal, higher chemical doses will be required to form a phosphate precipitate, and either filter loading rates must be decreased or, alternatively, an intermediate settling operation must be employed to handle the increased chemical floc production.

Common problems which have been encountered in direct, dual-media filtration relate to the difficulty of achieving economically long filter runs under the inherently high solids load coming onto the filter. Generally, filter runs will be shorter than those when an intermediate sedimentation step is provided, and, as a result, more backwash wastewater will be generated.

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However, there would probably be an equal amount of waste solids to dispose of if a combination sedimentation-filtration system were employed, so that solids disposal in a system using direct dual-media filtration should not be an unusually difficult problem.

For direct-dosing filter operations, the length of filter run and the rate of filter headloss development are usually directly related to the filter loading rate. As mentioned earlier, Conley and Hsiung (37) have found that filter loading rates of 5 gpm/ft$^2$ produced filter runs of 12 hours using 32 mg/l alum and 0.2 mg/l polymer as a filter aid on raw water treated for potable use. Slechta and Culp (38) found that loading rates of 5 gpm/ft$^2$ with an alum dose of 200 mg/l produced filter runs ranging from 2 to 8 hours when treating secondary effluent at the Lake Tahoe plant. The filter loading rate will, of course, be dependent to a large degree upon the quality of the wastewater being filtered, as well as the size of the media employed in the filter.

Organic polymers have contributed greatly to the effectiveness of direct dual-media filtration by acting to strengthen the weak chemical floc produced by the alum coagulation step. Higher filter flow rates are thus possible without shearing the fragile floc out of the filter bed. The usual polymer dose when used as a filter aid is about 0.1 mg/l (39)(40).

The process of direct filtration of alum-coagulated wastewater relies on the characteristic of the alum coagulant to undergo hydrolysis and then precipitate the solids and phosphates contained in the wastewater (41). The mechanism for surface charge reduction and subsequent agglomeration of suspended colloidal solids has been previously mentioned, but the mechanism for phosphate removal is still rather obscure. Despite considerable research, the basic chemistry of the phosphate reaction with Al(III) cations is quite unclear and the accumulated data has often been directly contradictory. To illustrate, Lea, Rolich, and Katz (42) presented data supporting the view that phosphate is removed by adsorption on precipitating aluminum hydroxides. However, Stumm (43), Cole and Jackson (44), Recht and Ghassemi (45), and Farrell, Salotto, Dean, and Tolliver (46), have all concluded that removal of phosphate takes place primarily by means of a chemical reaction rather than by adsorption. In general, it appears that adsorption on hydrated oxides appears to play a significant role along with the chemical formation of an insoluble particle in the removal of phosphate by alum. Clearly, further research is needed to give a true definition of the mechanism for phosphate removal using alum as a coagulant.

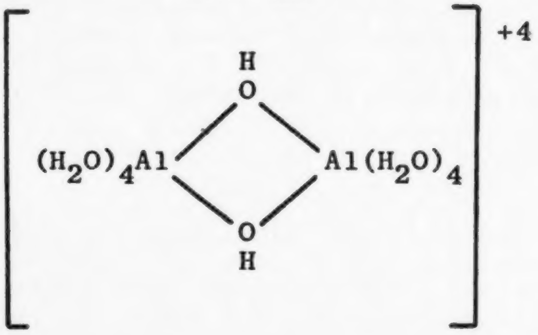
The literature also shows considerable disagreement among researchers concerning the kinetics and stoichiometry of the cation-phosphate reactions, as well as the effect of various parameters such as pH and ionic character of the wastewater on the removal of phosphate. Various researchers (39), (43), (47), have indicated that with proper pH conditions, and low cation-to-phosphate ratios, the chemical reaction for Al(III) cations with ortho-phosphate is

$$\mathrm{Al(III)} + \mathrm{H_2PO_4^-} \rightarrow \mathrm{AlPO_4} + 2\mathrm{H^+}$$

provided that adequate reaction time is provided. In practice, however, far more than the stoichiometric amount of Al(III) is required for complete precipitation of ortho-phosphates. Recht and Ghassemi (45) have indicated that amounts of Al(III) in excess of stoichiometric requirements can be satisfactorily explained in terms of the occurrence of competing reactions, such as

$$\mathrm{Al_2(SO_4)_3} + 6\mathrm{HCO_3^-} \rightarrow 2\mathrm{Al(OH)_3} + 6\mathrm{CO_2} + 3\mathrm{SO_4^{=}}$$

and the dispersion of metal-$\mathrm{PO_4}$ precipitate into extremely fine and often non-settleable colloidal particles. The required excess of Al(III) is thought to be due to the fact that hydrolysis products of Al(III) and not the Al(III) species alone are involved in the precipitate formation. According to Cohen and Hannah (40), these hydrolysis products may be transient monomeric species such as $[\mathrm{Al(H_2O)_4(OH)_2}]^{+1}$ or more complex polynuclear species such as $[\mathrm{Al_8(OH)_{20}}]^{+4}$ or



Positively charged hydroxy-aluminum polymers are also indicated by Hsu (48), and Yuan and Hsu (49) to be the species accounting for the precipitation of phosphate. These authors have stated that this precipitation process is greatly influenced by the ratio of phosphate to aluminum and also by the nature and concentration of foreign components present. The precipitation will be either aided or hindered, depending on whether there are just enough foreign anions to help the $PO_4$ anions neutralize the positive hydrozy-aluminum polymers, or whether there are so many foreign anions present that competition exists between them and $PO_4$. Recht and Ghassemi (45) also showed that an approximate linear relationship exists between the residual ortho-phosphate concentration and the Al/Ortho-$PO_4$ molar ratio up to a ratio of about 1:1 in a pure ortho-phosphate solution at a pH of 6.0. Their work indicated that 1.4 moles of Al(III) are required to precipitate one mole of $PO_4$. This linear relationship at fairly low Al/$PO_4$ molar ratios suggests the occurrence of a chemical reaction which is dependent on the molar quantities in solution.

## EXPERIMENTAL PROGRAM

The objectives of the pilot plant study on the high rate clarification system were to determine the optimum chemical doses and operating conditions for two situations: 1) clarification only, and 2) clarification plus high $PO_4$ removal. In each case various constituent concentrations were monitored (COD, BOD, bacteria, trace metals) to determine the overall effectiveness of each treatment case. It has been shown that the chemical doses required for solids removal are lower than those necessary for $PO_4$ removal (50). Thus, it is important to determine the relative doses required so that efficient plant operation can be maintained under various required effluent qualities. For production of a reusable water, the required advanced waste treatment plant effluent quality may indicate a low ammonia content (provided by a nitrification unit, for example) and very low suspended solids content. In this case, a lower alum dose will be required to coagulate and remove the solid particles than would be necessary for $PO_4$ removal.

## RESULTS

As with the conventional clarification system, one-liter samples were composited over a four hour period after equilibration of the system. For the high rate system, the points sampled were 1) the pilot plant influent, 2) the bio-clarifier effluent, and 3) the filter effluent. Analysis of these samples included the same range of variables evaluated for the conventional system.

The wastewater feed to the alum system consisted of effluent from the Denver Sewage Treatment Plant which had been nitrified in the preceding pilot plant unit. This wastewater feed displayed widely fluctuating constituent concentrations. Since this was a pilot plant study and the emphasis was on simulating a full-scale operation, the chemical doses were held constant for the length of each run and no attempt was made to measure the changing influent and pace the chemicals accordingly. Because of this, the results show the type of variation that might be more characteristic of plant scale operations than the controlled uniform input research studies.

## Flocculating Characteristics

One of the initial investigations in the high rate system was an analysis of the effect of slow mix paddle speed and detention time on the removal efficiency of the dual-media filter. Mixing speeds of 7 rpm, 15 rpm, and 23 rpm were investigated over a period of several weeks. A careful study of the data showed that, although the physical appearance of the chemical floc was quite different at the various mixing speeds (large floc at low speeds, small floc at high speeds), the phosphate, suspended solids, COD, BOD, and turbidity removals were not significantly affected by changes in the slow mix paddle speed within the range of 7 to 23 rpm. The 7 rpm paddle speed allowed for larger floc particle formation due to the lower mixing intensity, but in the transmission to and through the filter the large, weak floc particles were reduced to a much smaller size. This floc was similar to the floc produced in the slow mix unit at the higher mixing intensities.

The slow mix detention time varied from 4 minutes to 60 minutes, and an analysis of the data gathered at the different detention times suggests that the various detention times neither aided nor hindered the removal of pollution constituent concentrations over the detention time range that was studied. However, it is clear that if the same results can be achieved at 4 minutes detention time as those at 60 minutes detention time, then a much smaller slow mix unit can be utilized in the treatment process for the shorter detention time. In fact, Conley and Hsiung (37), and Slechta and Culp (38) have reported that the best clarification and phosphate removal may be achieved with partically no flocculation period, other than that which may occur during the subsequent filtering process itself. At the South Lake Tahoe facility, Slechta and Culp (38) simply added the alum coagulating chemical directly to the filter influent thereby forming a "micro" floc which effectively removed suspended solids and phosphates.

## pH Variation

A wastewater parameter in the high rate clarification process which is very important to the system success is the pH. There have been several discrepancies reported in the literature for the optimum pH of aluminum phosphate precipitation. Lea, Rolich, and Katz (42) have reported an optimum pH range of 7.1-7.7, while Cohen and Hannah (40) indicated a pH range of 5.5-7.8. Recht and Ghassemi (45), and Leckie and Stumm (51) confirmed the middle of this latter pH range with optimums of pH 6.0 and 6.3, respectively. Of course, the precise optimum for phosphate removal will vary somewhat with the actual chemical constituents comprising the wastewater. Data are shown in Figure 46 for the pH investigation which covered a pH range from 3.0 to 8.7. It can be seen from these results that the optimum pH for alum treatment at the pilot plant covers the range of 5.0-6.0. The residual suspended solids and BOD levels remain quite low throughout the lower pH range, but begin to rise above a pH of about 6. This reduction in removal efficiency is attributable to the less efficient coagulation at the higher pH values. Figure 46 includes only those alum doses for which the $Al/PO_4$ molar ratio is greater than 1.0. This was done to reduce the direct, linear effect that relatively low mole concentrations of aluminum have on phosphate removal (46).

## Alum Dose Variation

The removals of phosphates, suspended solids, COD, BOD, and turbidity as they vary with the applied alum dose are shown in Figures 47 through 54. When it was found that the slow mix paddle speed and detention time were not major parameters in the alum system, data at all paddle speeds and detention times were included in these plots of alum dose vs. constituent residuals.

Presented in Figure 47 is a plot of ortho-phosphate removal vs. alum dose. It can be seen from this curve that about 120 mg/l alum is required to lower the residual ortho-phosphate to one mg/l. This corresponds to approximately 96% ortho-phosphate removal. Also shown on Figure 47 are the results of jar tests which were conducted on Denver Metro secondary effluent at the beginning of the study. The jar test results and the pilot plant results are in close agreement, although the jar tests incorporated 30 minutes of settling while the pilot plant flow underwent direct dual-media filtration to remove the accumulated chemical floc.

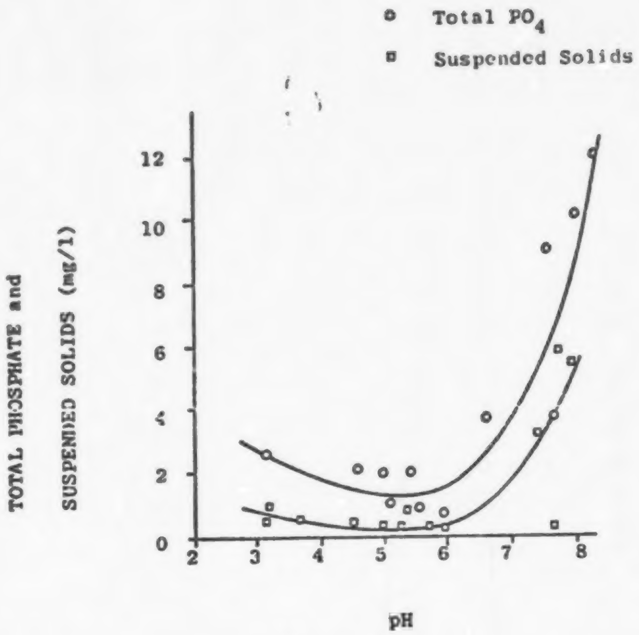


Figure 46

pH OPTIMIZATION FOR THE HIGH RATE ALUM SYSTEM

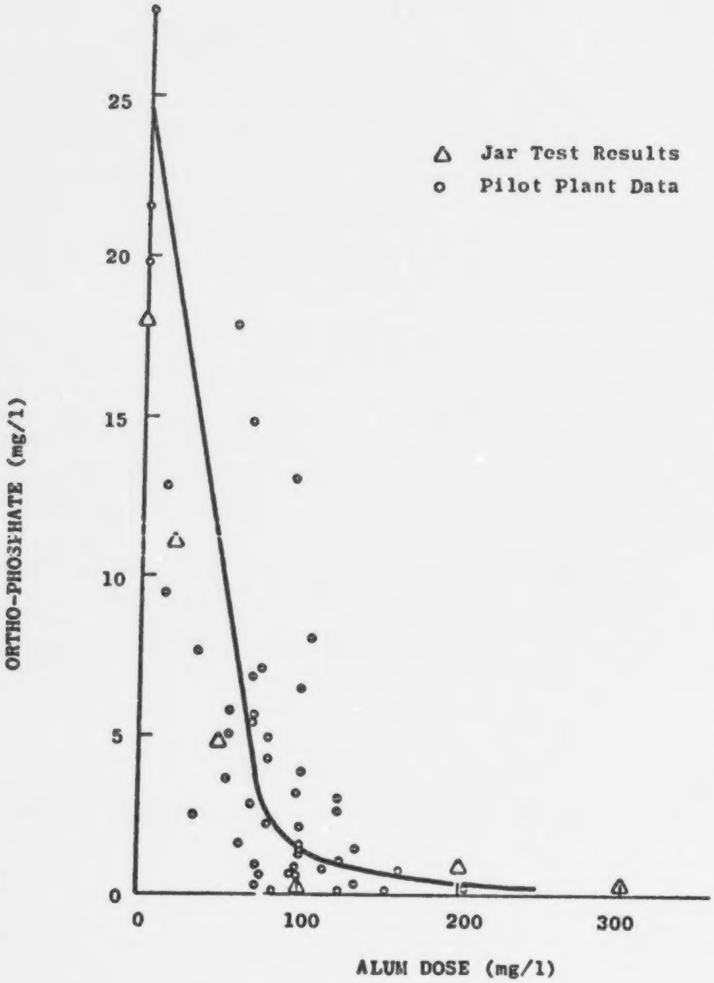


Figure 47

ORTHO-PHOSPHATE REMOVAL WITH ALUM

Figure 48 represents data on total phosphate residual vs. alum dose. Data points include residuals from dual-media filtration alone, as well as residuals from combined dual-media and membrane filtration. As shown by the curve, for a particular filter run, the dual-media filtered effluent generally contained slightly more residual total phosphate than the membrane - filtered sample. The agreement between the two curves is an indication, however, that the dual-media filter actually retained nearly all of the insoluble phosphate in the wastewater. The definition of insoluble phosphate for this purpose is that which will not pass a 0.45 micron membrane filter.

From Figure 48 it can be seen that a dose of about 120 mg/l alum was required to consistently produce a residual total phosphate level of about one mg/l. It should be noted that an increase in the alum dose from 110 mg/l to 200 mg/l, an increase of 82%, increased total phosphate removal by only 5%.

The scatter of data in Figures 47 and 48 is primarily due to the variation in phosphate concentrations entering the high rate system. These variations arise from fluctuations in soluble phosphorus levels and the significant amounts of organic biomass containing phosphorus which periodically passed through the nitrification clarifier and into the alum system. These variations have caused the total phosphate concentrations to range from 12.2 mg/l to 38.8 mg/l, with an average of 23.3 mg/l as $PO_4$.

In Figure 49 the residual total phosphate data from Figure 48 has been plotted against the $Al/PO_4$ molar ratio. In this plot the wide-ranging influent phosphate concentrations can be accounted for by ratioing them with the particular alum dose for that day. This appears to be a valid way of comparing phosphate removals at various initial phosphate concentrations when using low doses of alum. At low alum doses the chemical nature of the aluminum-phosphate reaction predominates and the alum requirement corresponds much more closely to stoichiometric relationships than it does at higher alum doses (46).

It is indicated in Figure 50 that turbidity in the final filtered effluent was reduced to approximately 0.5 mg/l $SiO_2$ by addition of an alum dose of 50 mg/l. Influent turbidities ranged from 0.4 mg/l to 49 mg/l as $SiO_2$. This represents an overall turbidity removal of about 97% with both dual-media and rapid sand filtration. These removal efficiencies did not improve significantly above an applied alum dose of 50-60 mg/l.

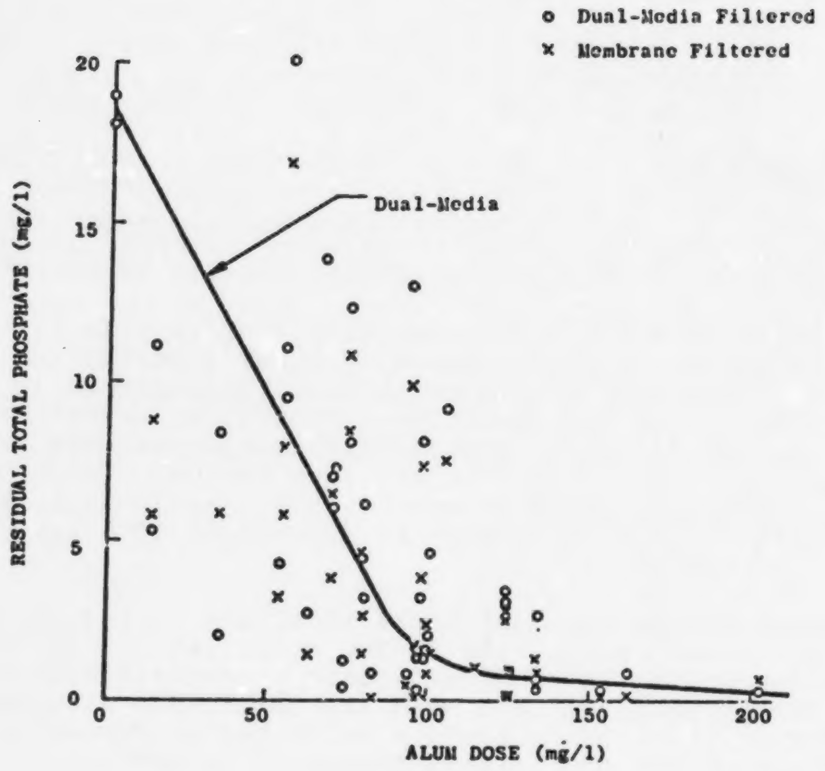


Figure 48

TOTAL PHOSPHOROUS REMOVAL WITH ALUM

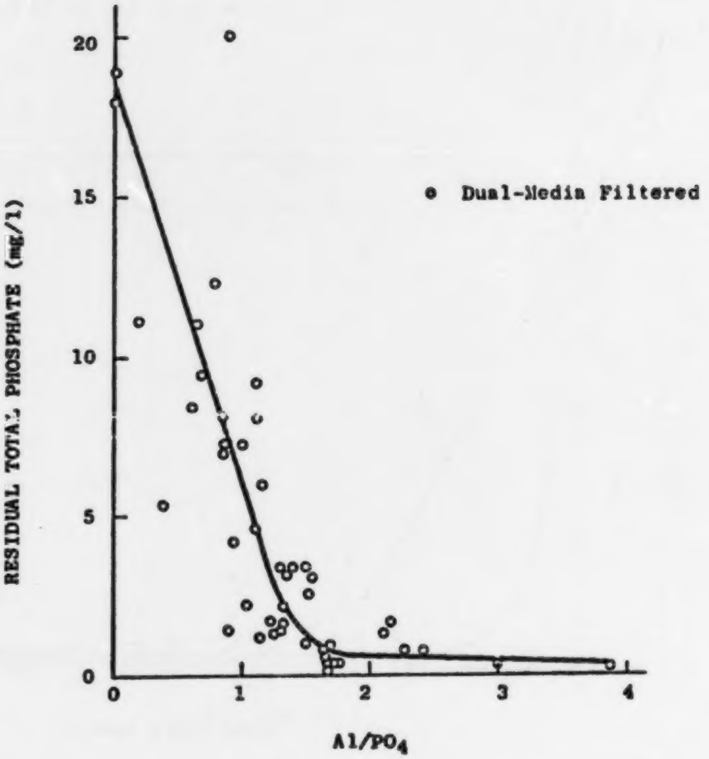


Figure 49

TOTAL PHOSPHATE REMOVAL AS A FUNCTION OF Al/PO$_4$ MOLAR RATIO

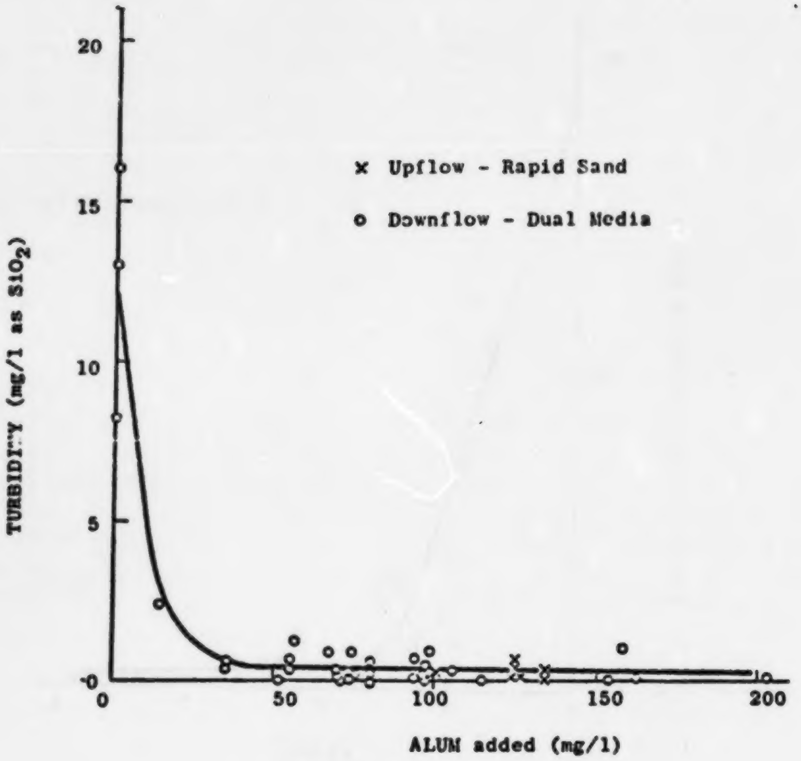


Figure 50

TURBIDITY REMOVAL WITH ALUM

Suspended solids removal is shown in Figure 51 as a function of alum dose. During the testing period the influent suspended solids at point two in the alum system averaged 41 mg/l, with a high of 108 mg/l and a low of 8 mg/l. A much lower alum dose was required for effective clarification than was required for phosphate removal, as only about 50-60 mg/l alum was necessary to achieve a suspended solids residual of less than one mg/l. During a portion of the study, the alum-coagulated wastewater was filtered through the rapid sand filter at the pilot plant in both upflow and downflow operational modes. Suspended solids removal was quite good at all times (< 1 mg/l residual) as shown by the "x" data points in Figure 51.

The high rate clarification system was effective in removing the suspended fraction of the COD. The COD influent ranged from 28.8 mg/l to 174 mg/l, with an average of 78.3 mg/l. The average residual COD shown in Figure 52, was about 20 mg/l, representing the soluble refractory organics. The applied alum dose did not appear to significantly affect COD removal at alum doses of greater than 50 mg/l. Results indicate that about 75% of the COD had been removed at an alum dose of 50-60 mg/l. Again, data for COD removal by rapid sand filtration is similar to that from dual-media filtration.

BOD is another parameter which varied widely in the alum system influent, ranging from a high of 159 mg/l to a low of 3.5 mg/l. Figure 53 indicates that reductions to 2 mg/l BOD were achieved at alum doses as low as 50-60 mg/l. Increasing the alum dose above this amount did not produce significant improvements in removal. Filtration through dual-media and rapid sand produced almost identical results - removals of about 98%.

The last graph in this series, Figure 54, combines the phosphate, suspended solids, COD, BOD, and turbidity residuals plotted as a function of the applied alum dose. This graph clearly shows that less alum can be used for effective clarification than for effective phosphate removal.

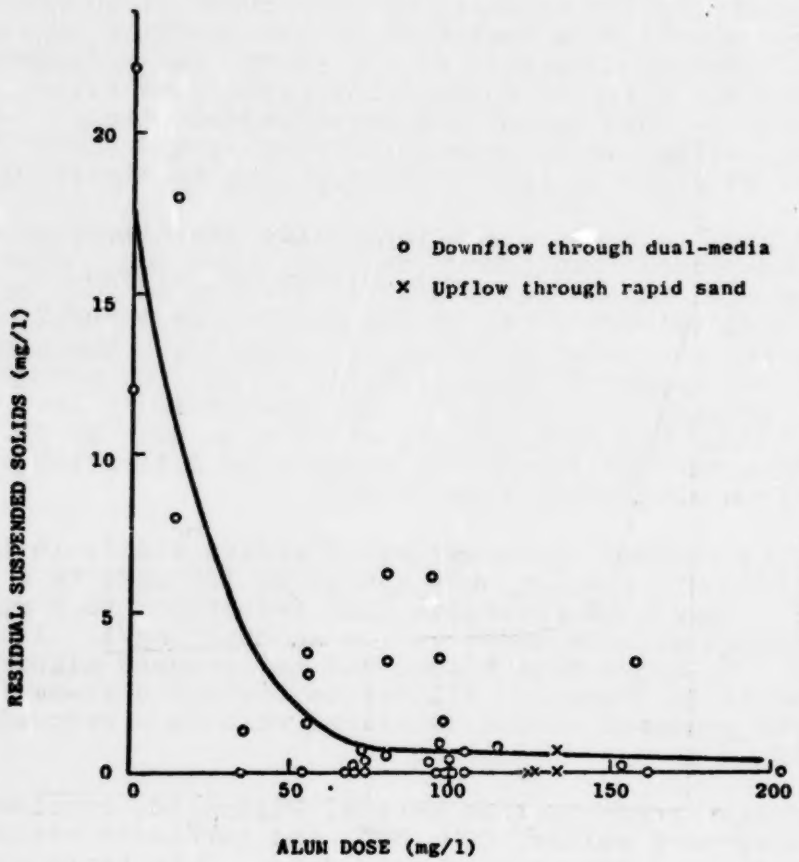


Figure 51

SUSPENDED SOLIDS REMOVAL WITH ALUM

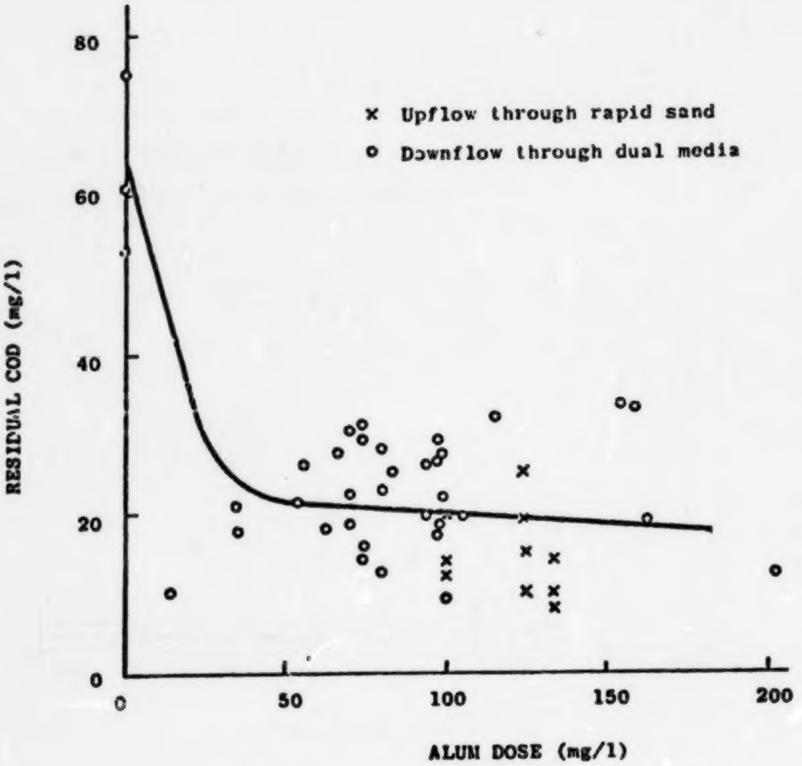


Figure 52

COD REMOVAL WITH ALUM ADDITIONS

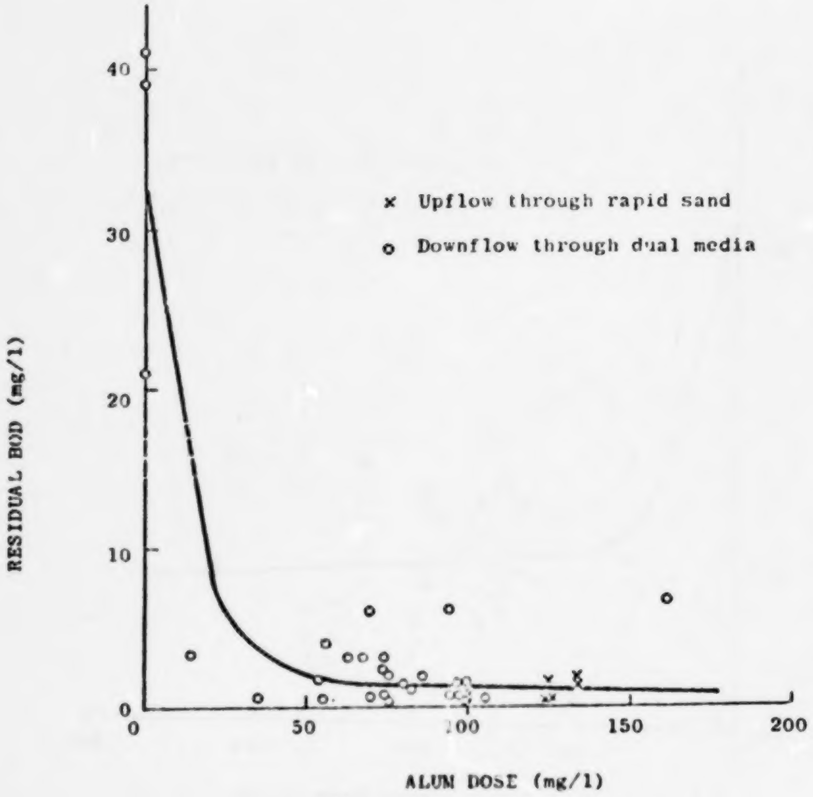


Figure 53

BOD REMOVAL WITH ALUM

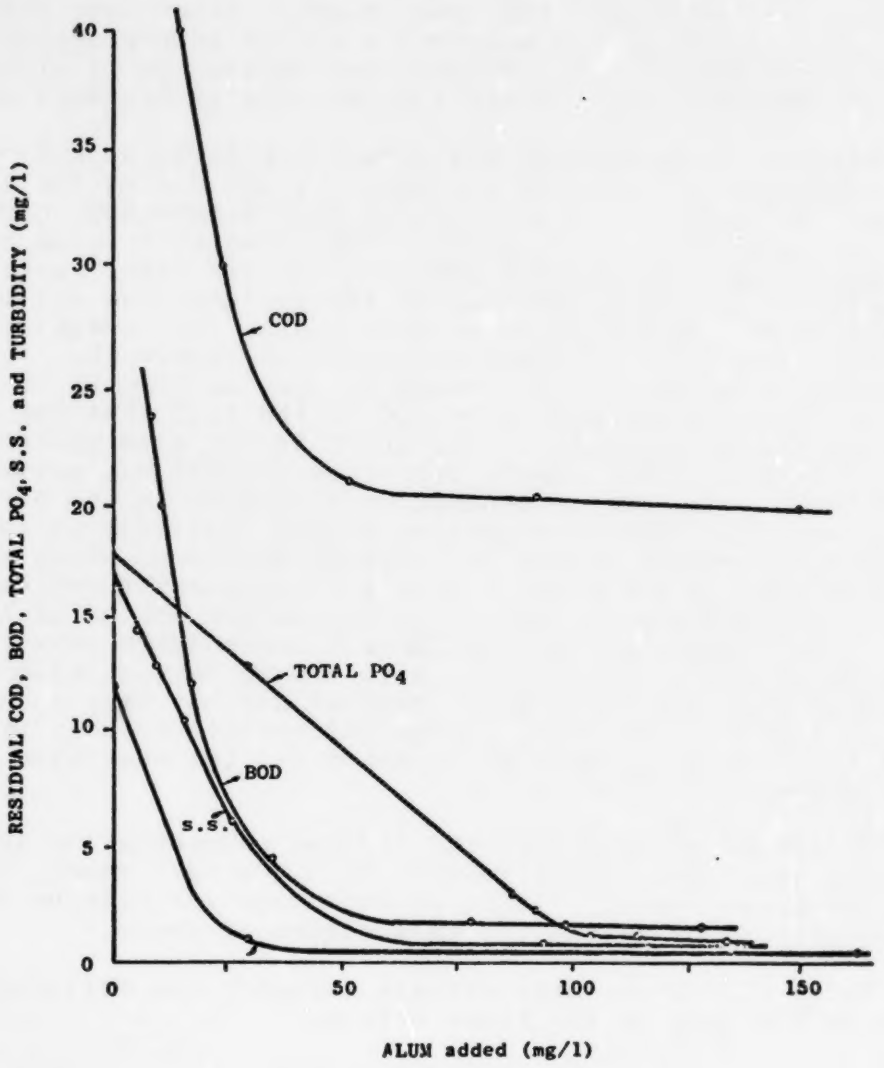


Figure 54

RESIDUAL POLLUTIONAL PARAMETERS AS A FUNCTION OF ALUM DOSE IN THE HIGH RATE SYSTEM

## Trace Metal Removals

As performed for the conventional system, trace metal analyses were conducted for iron, copper, manganese, chromium, lead, zinc, cadmium, and molybdenum by the laboratory of the Denver Water Board. All analyses were conducted by atomic absorption methods given in the 12th edition of Standard Methods.

Figures 55 through 63 illustrate the influent and effluent concentrations for each of the trace metals during the testing period. It is shown on the figures that molybdenum, cadmium, zinc, and lead were not significantly affected by alum treatment while some removal was observed for the other trace metals as indicated by the separation of the influent and effluent average concentration lines on each graph. An exception to the above statement is shown in Figure 56, where the copper concentration consistently increased in passing through the alum system. This is thought to be due to the fact that the aeration pipes in the nitrification unit prior to the alum system were made of copper, which, under corrosive conditions, naturally acted to increase the concentration of copper in the surrounding wastewater. Other sources of copper initially in the alum system were several valves and hose connections, which were replaced late in the study. This may explain the decreasing copper residual concentrations in the latter months of the project. For copper, as well as for all other trace metals, an analysis of the daily results, with widely varying alum doses, suggested that the trace metal removal did not vary as a function of applied alum dose. Large differences in residual trace metal concentrations were often noted for the same alum dose application.

Figure 61 reveals that the cadmium concentration in the effluent was consistently reduced to 0.002 mg/l which, as seen from the graph, was the limit of detection for cadmium by the atomic absorption method for this series of tests.

Table VIII represents average influent and effluent concentrations for each of the trace metals.

## Bacteriological Analyses

The Water Laboratory of the Denver Water Board monitored coliforms, fecal coliforms, and fecal streptococcus on a weekly basis throughout the duration of the pilot alum study. The variation of influent and effluent bacterial concentration with time is shown in Figures 64, 65, and 66. Generally, a reduction in coliforms of greater than 97% was achieved by alum treatment irrespective of the alum dose. This is indicated with selected data included in Table IX.

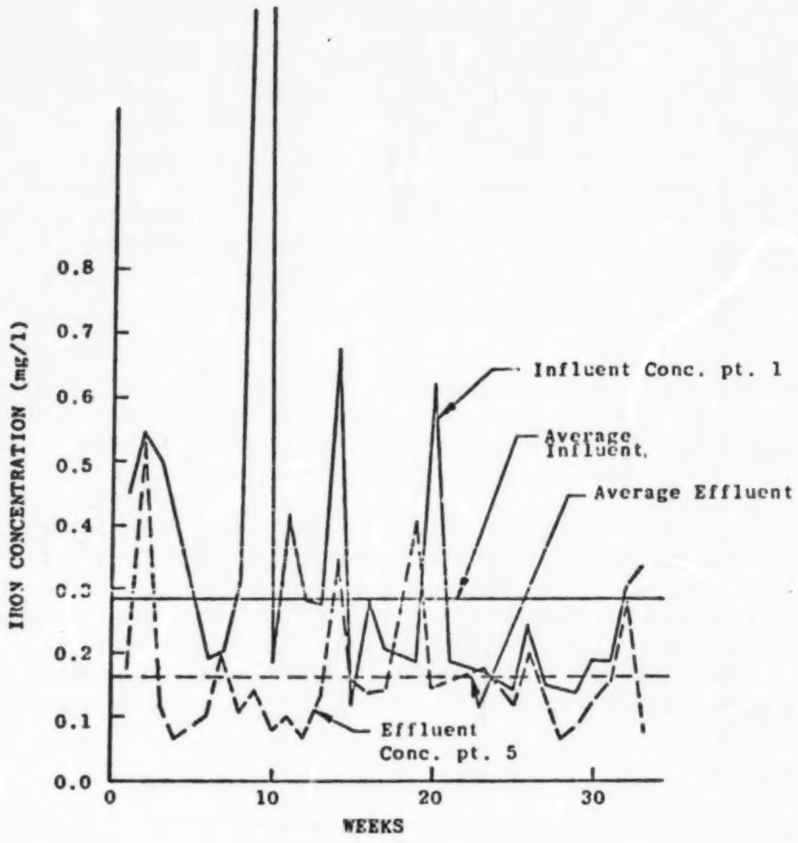


Figure 55

INFLUENT AND EFFLUENT IRON
CONCENTRATIONS

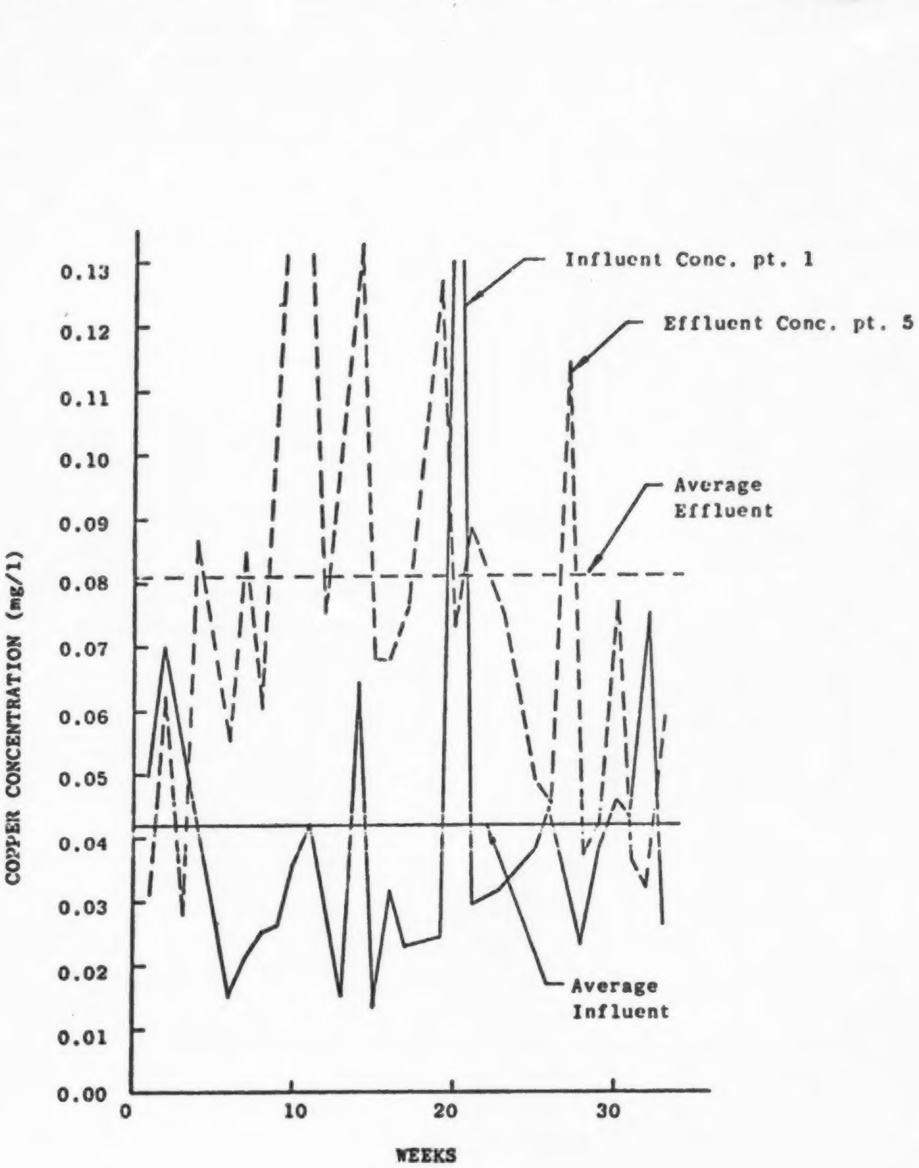


**Figure 56**

INFLUENT AND EFFLUENT COPPER CONCENTRATIONS

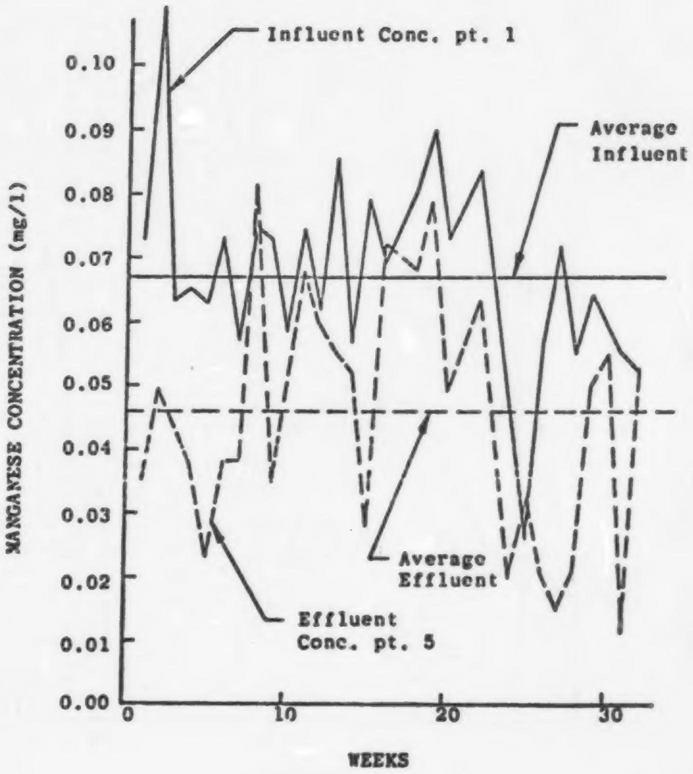


**Figure 57**

INFLUENT AND EFFLUENT MANGANESE CONCENTRATIONS

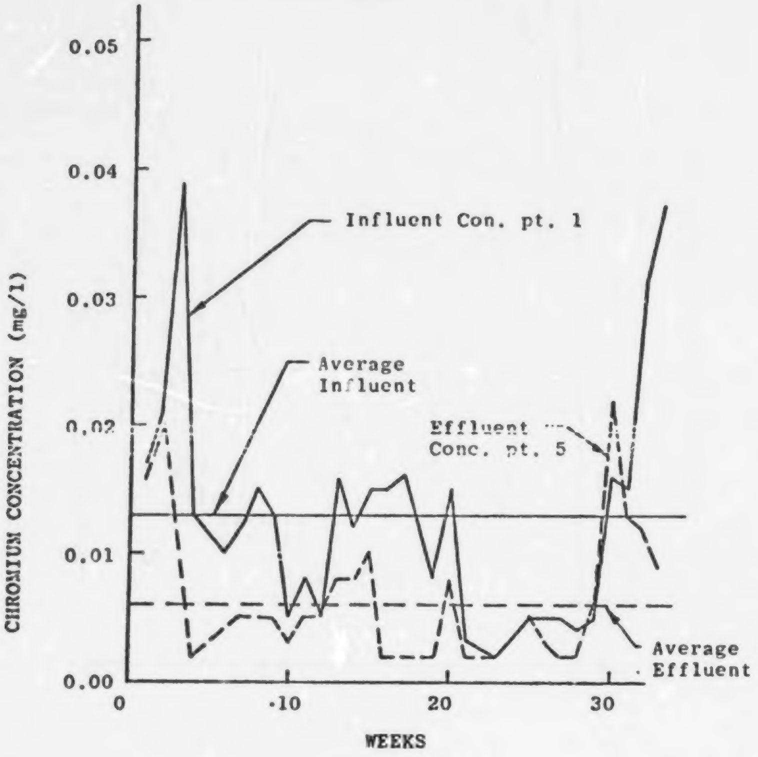


Figure 58

INFLUENT AND EFFLUENT CHROMIUM CONCENTRATION

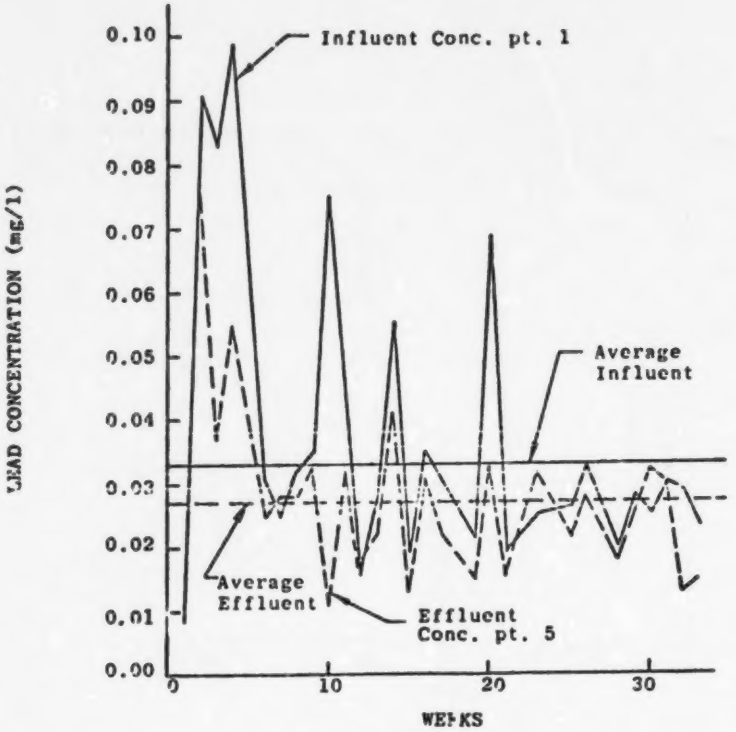


Figure 59

INFLUENT AND EFFLUENT LEAD CONCENTRATION

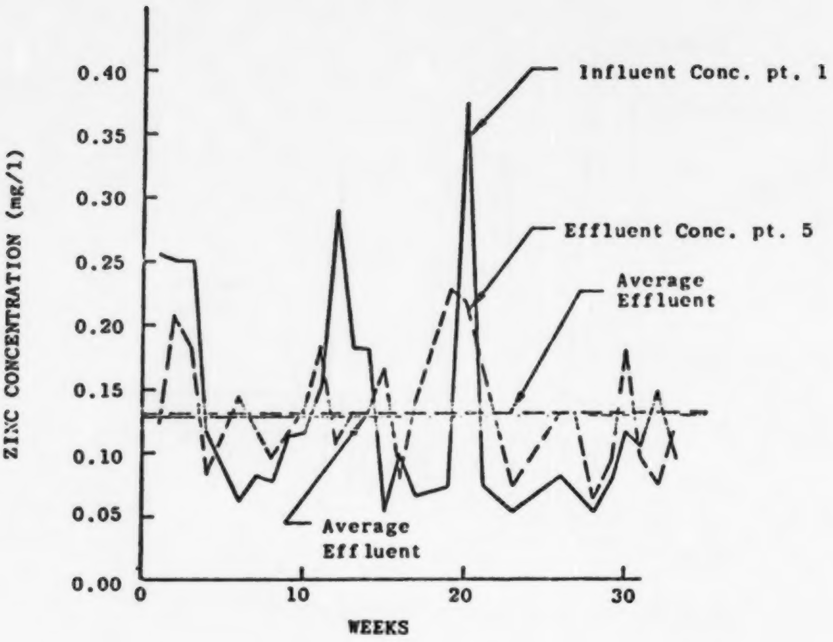


**Figure 60**

INFLUENT AND EFFLUENT ZINC CONCENTRATION

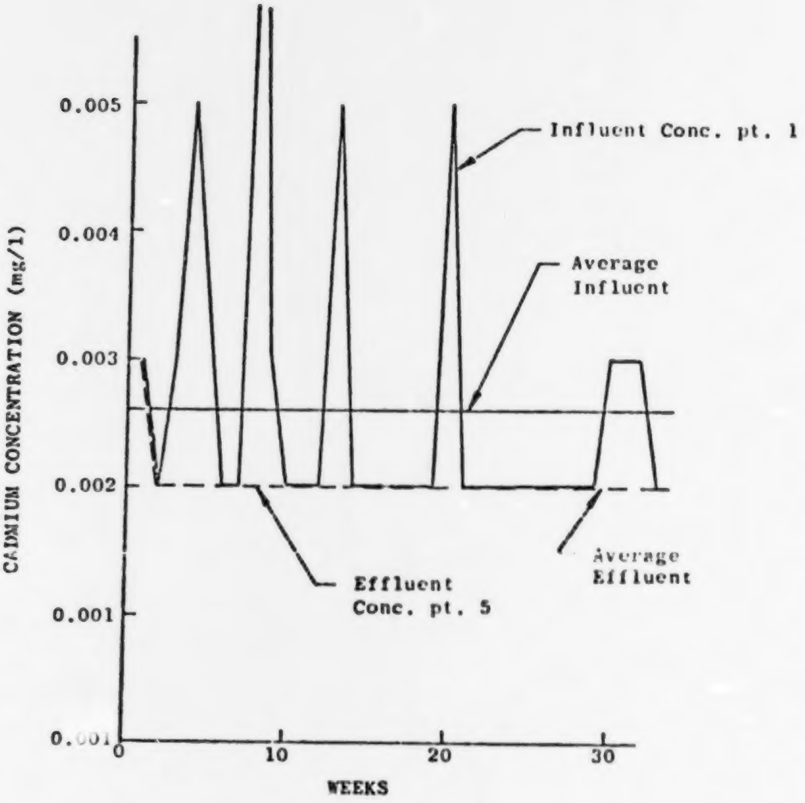


Figure 61

INFLUENT AND EFFLUENT CADMIUM CONCENTRATION

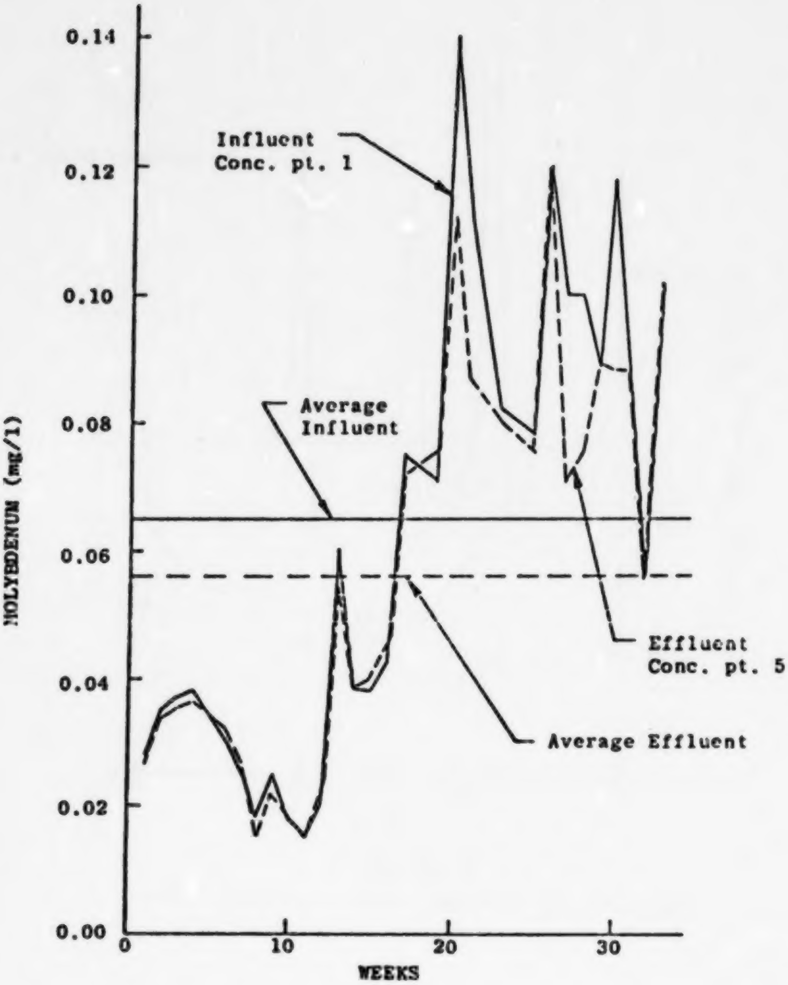


Figure 62

INFLUENT AND EFFLUENT MOLYBDENUM CONCENTRATIONS

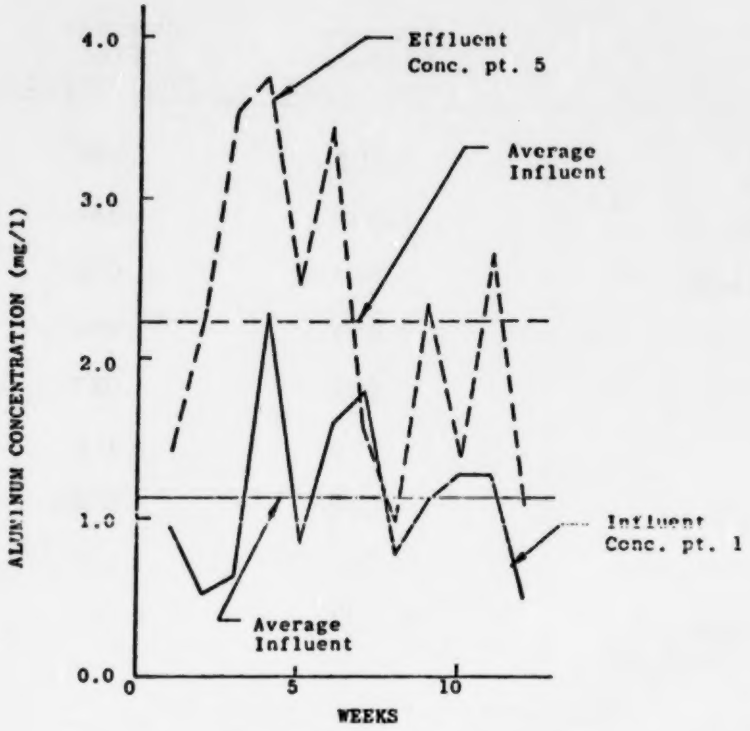


Figure 63

INFLUENT AND EFFLUENT ALUMINUM CONCENTRATION

## Table VII

### Average Trace Metal
### Influent and Effluent Concentrations

| Metal | Average Influent Conc. mg/l | Average Effluent Conc. mg/l | Average % Removal |
|---|---|---|---|
| Iron | .285 | .162 | 43.2 |
| Copper | .042 | .081 | +92.8 |
| Manganese | .067 | .046 | 31.3 |
| Chromium | .013 | .006 | 53.8 |
| Lead | .033 | .027 | 18.2 |
| Zinc | .129 | .130 | + 0.8 |
| Cadmium | .0026 | .0021 | 19.2 |
| Molybdenum | .065 | .056 | 13.8 |
| Aluminum | 1.124 | 2.233 | +98.7 |

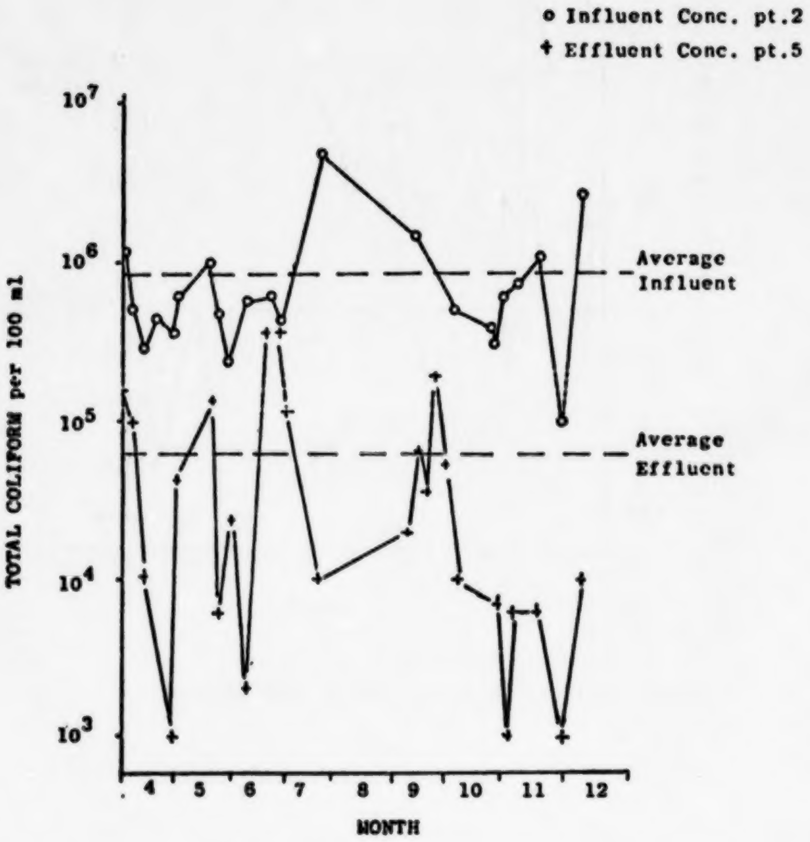


**Figure 64**

TOTAL COLIFORM REMOVAL HISTORY WITH ALUM

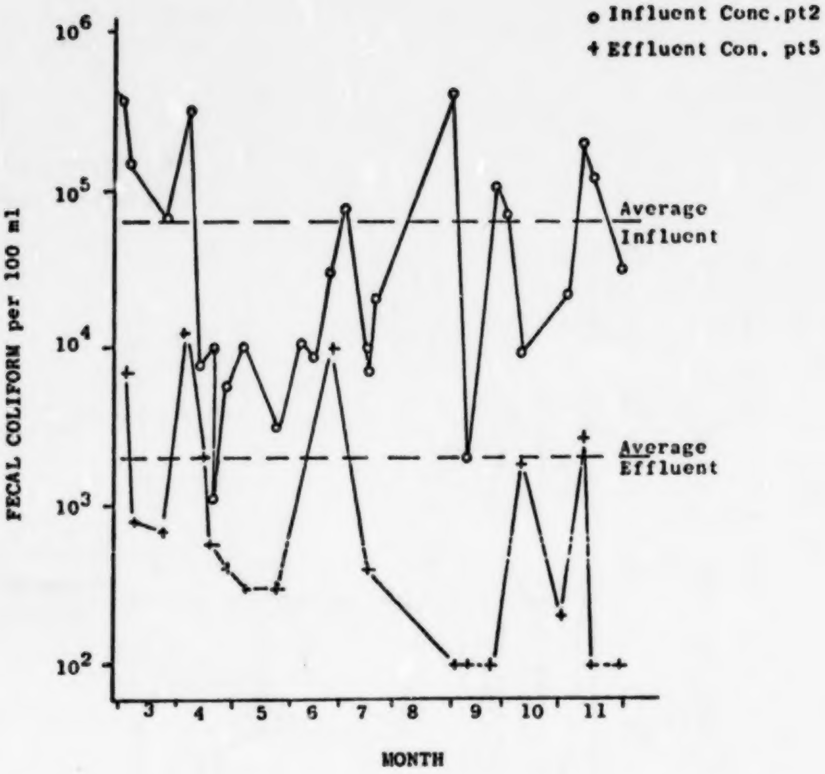


**Figure 65**

FECAL COLIFORM REMOVAL HISTORY WITH ALUM

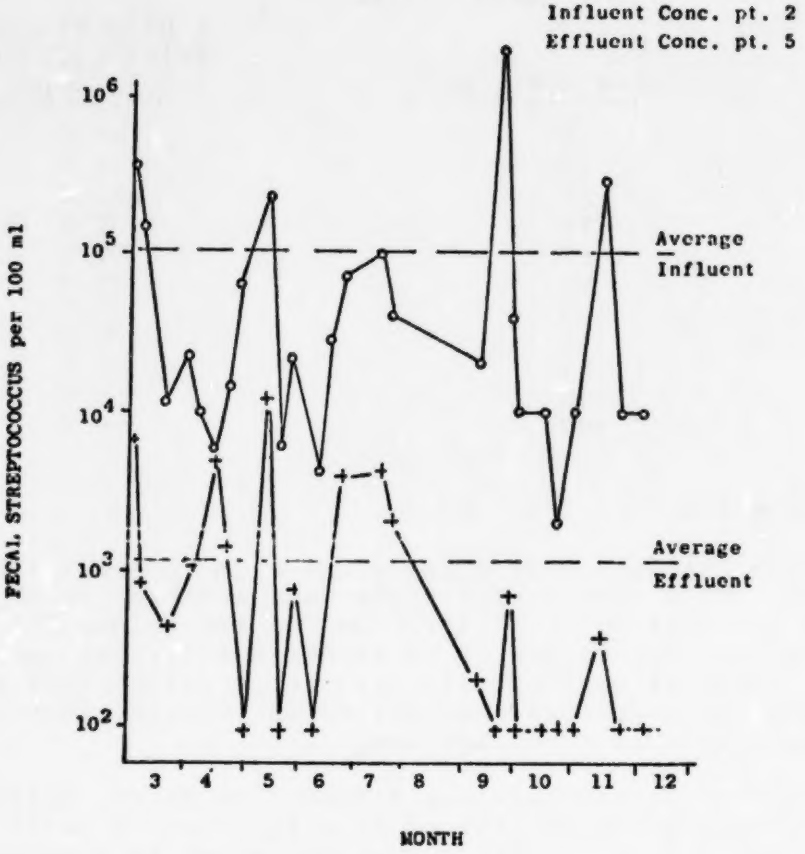


Figure 66

FECAL STREPTOCOCCUS REMOVAL HISTORY WITH ALUM

## TABLE VIII

### Alum Dose vs Percent Reduction of Total Coliforms

| Date | Alum Dose, mg/l | % Reduction of Total Coliforms (pt. 2 - pt. 6) |
|---|---|---|
| 4-8-71 | 0 | 80.0 |
| 7-15-71 | 14 | 99.7 |
| 7-6-71 | 35 | 100.0 |
| 8-3-71 | 74 | 97.8 |
| 4-27-71 | 80 | 97.6 |
| 9-28-71 | 94 | 98.0 |
| 5-20-71 | 100 | 98.8 |
| 5-25-71 | 115 | 98.8 |
| 6-1-71 | 154 | 99.8 |
| 4-20-71 | 168 | 98.9 |
| 6-10-71 | 203 | 99.6 |

### Filter Flow Rates

In-depth filtration is a key element in the efficacy of this system. This type of filtration was achieved in this study with the dual-media filter. One of the primary factors influencing the performance of an in-depth filter is the filter flow rate. This is particularly true for a filter that must process chemically-treated, nonsettled wastewater, such as that produced in this investigation.

In order to compare various filter flow rates, filter headloss curves have been plotted as a function of total solids capture in the filter. These curves are shown in Figures 67 through 72. Each figure shows several headloss curves which were produced at a constant filter flow rate. The five figures, 67 through 71, show headloss curves at filter flow rates of 2.3, 3.0, 3.8, 5.0, and 5.7, all expressed as gpm/ft$^2$.

The data are presented as headloss vs solids captured in grams/ft$^3$ rather than headloss vs time because of the great variation in influent suspended solids coming onto the filter. These solids are the primary cause of headloss development in a filter

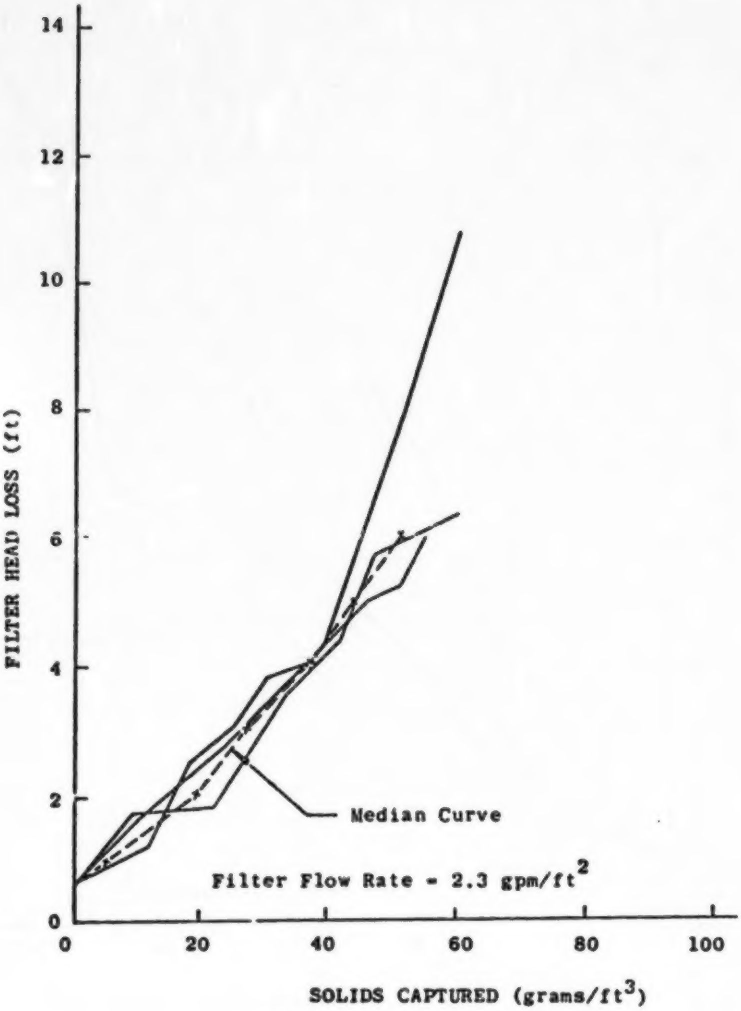


Figure 67

FILTER HEAD LOSS AS A FUNCTION OF SUSPENDED SOLIDS CAPTURED

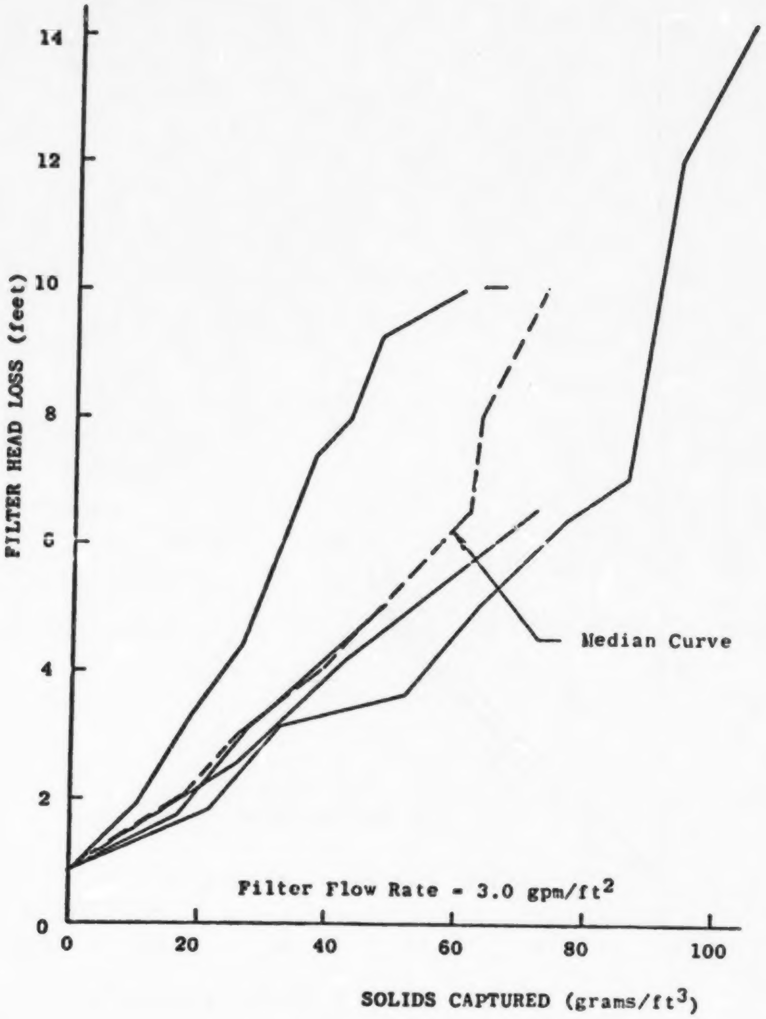


Figure 68

FILTER HEAD LOSS AS A FUNCTION OF SUSPENDED SOLIDS CAPTURED

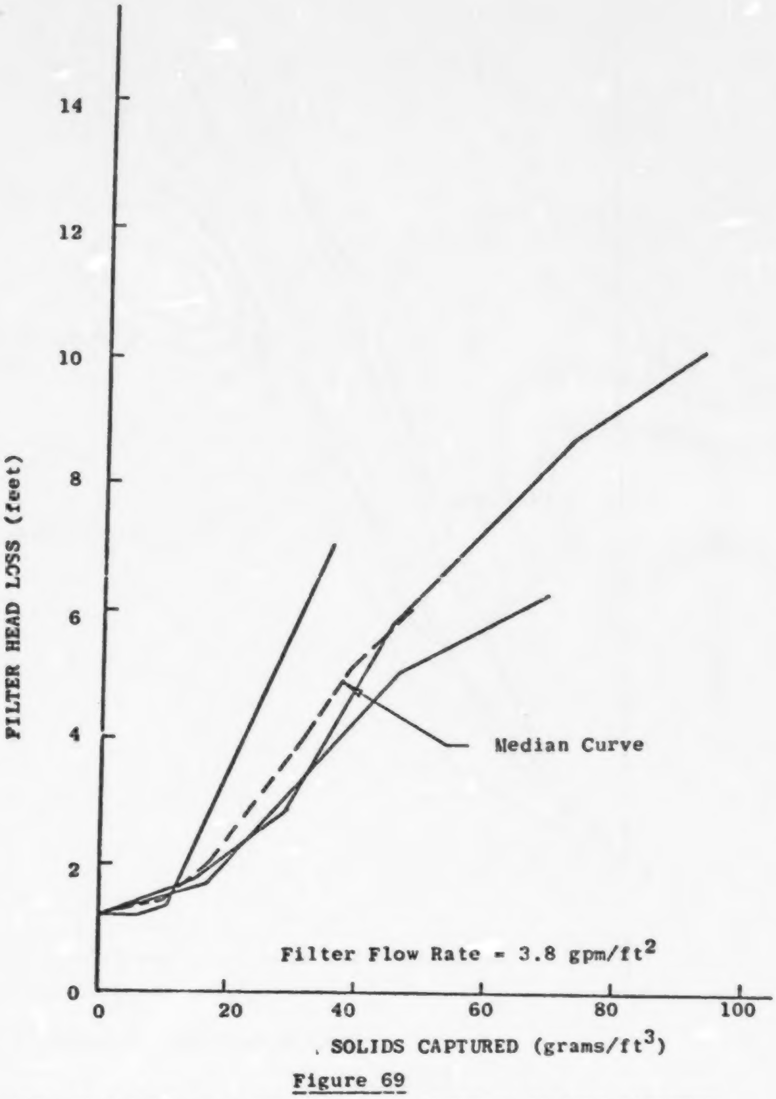


Figure 69

FILTER HEAD LOSS AS A FUNCTION OF SUSPENDED SOLIDS CAPTURED

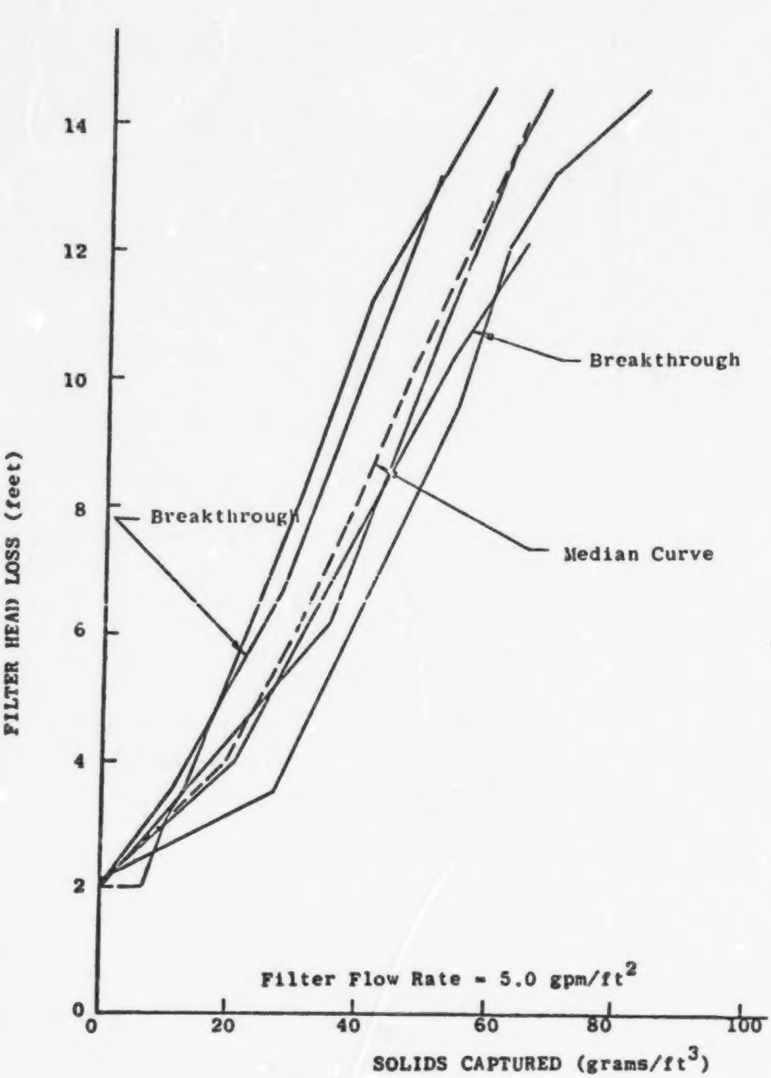


Figure 70

FILTER HEAD LOSS AS A FUNCTION OF SUSPENDED SOLIDS CAPTURED

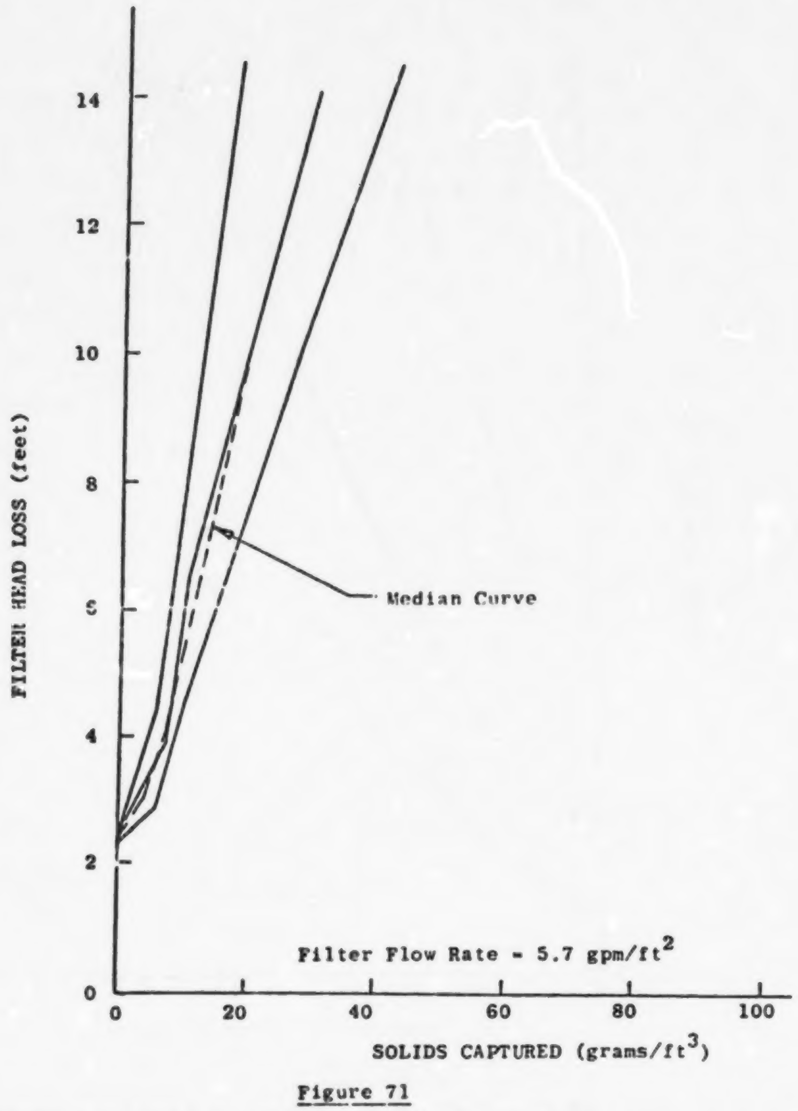


Figure 71

FILTER HEAD LOSS AS A FUNCTION OF SUSPENDED SOLIDS

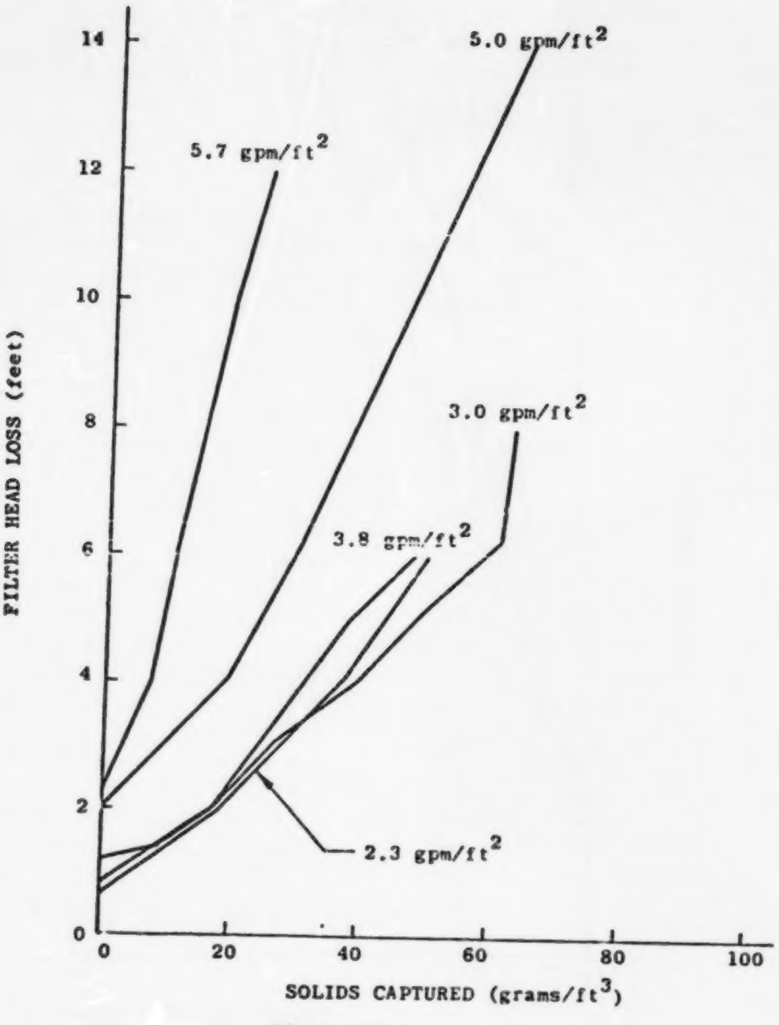


Figure 72

FILTER HEAD LOSS AS A FUNCTION OF SUSPENDED SOLIDS CAPTURED AND FLOW RATE at most flow rates due to their plugging effect and restriction of fluid flow through the filter. Therefore, a plot of filter headloss vs solids captured can incorporate both wide ranges in filter flow rates and large variations in filter influent suspended solids. This plot also helps to give a clearer picture of the comparative efficiencies of a filter operated at a constant flow rate and receiving wide-ranging concentrations of influent suspended solids during each of several days of filter operation. The alum dose during these runs was approximately 100 mg/l with an accompanying polymer dose of about 0.2 mg/l.

In each of Figures 67 through 71 a median curve has been plotted to represent the typical headloss curve for that particular flow rate. Each of these median curves is reproduced in Figure 72 to show the effect of filter flow rate on filter headloss. As expected, the higher flow rates produced higher headlosses than the lower flow rates. This difference in headloss is quite significant when comparing flow rates of 3.8, 5.0, and 5.7 gpm/ft$^2$ in this study. At flow rates ranging from 2.3 to 3.8 gpm/ft$^2$ the headloss development rate did not vary significantly. This illustrates the effect the higher filter flow rates have in breaking up the floc particles in the upper, coarse layers of the dual-media filter and carrying them further down into the smaller pore areas of the filter bed where they cause increased filter headloss because they then block off many of the smaller pore openings. Combined with this increased headloss at higher filter flow rates is the increasing head required to produce increasing flow-through velocities in any granular filter, whether clean or clogged.

At the higher filter flow rates there will also be the tendency for the hydraulic gradients to wash the solid particles caught in the filter pores completely out of the filter. This tendency must be compensated for by adding a polyelectrolyte filter aid to the filter influent. This helps to strengthen the chemical floc and keep it in the filter. In this research the slightly anionic acrylamide polymer "Separan" a product of Dow Chemical Company, has been found to be effective in the dose of range 0.1 to 0.3 mg/l.

Figure 73 is a plot of filter headloss vs flow rate in gpm/ft$^2$ for clear water flowing through the dual-media filter. The curve indicates the hydraulic head that is produced at each of the various filter flow rates, and this head is shown in Figures 67 through 72 as the point when solids capture began (filter run was initiated).

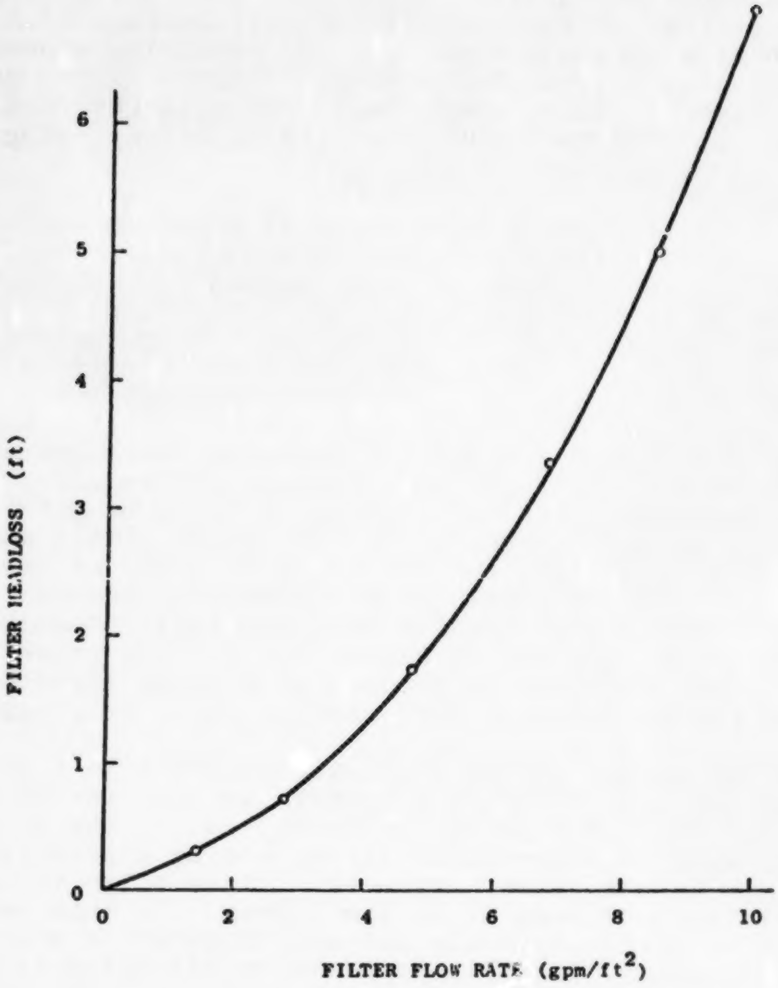


Figure 73

HEADLOSS AS A FUNCTION OF FLOW RATE FOR CLEAR WATER FLOWING THROUGH THE DUAL-MEDIA FILTER

The dual-media filter generally produced runs of acceptable effluent quality of 5 to 6 hours duration at a filter flow rate of 3 gpm/ft$^2$ and a polymer dose of about 0.2 mg/l. When the flow rate was increased to 5 gpm/ft$^2$ the duration of run was reduced to about 2 to 3 hours because of rapid headloss development. All filter runs were terminated either when effluent turbidity measurements showed that solids breakthrough was occurring, or when the headloss reached 14.5 feet, which was the limit for the unit.

# IX. ACKNOWLEDGEMENTS

Numerous organizations and individuals made highly significant contributions to the conduct of this investigation. Of particular note was the input provided by the staff of the Denver Water Department to the design, operation, and analyses of the pilot plant system. Individuals responsible for much of this cooperation were: J. L. Ogilvie, K. J. Miller, R. C. McWhinnie, S. W. Work, R. L. Heaton, W. R. Van Nattan, J. C. Dice, C. G. Farnsworth, Marsha Heinig, and John Akin.

Valuable assistance was also provided by the staff of the Denver Metropolitan Sewage Disposal District No. 1. Bill Korbitz, Joe Woodley, John Puntenney, William Martin and Harry Harada were instrumental in coordinating this assistance.

Several students at the University of Colorado were actively involved throughout the program in the sampling and analysis of the pilot plant system. These included: Doug Merrill, Bob Fox, Kamlesh Shah, Anthony Koltuniuk, and Ben Harding.

Helpful review and discussion of the project progress was provided throughout the duration of the investigation by E. F. Barth, the Project Officer for EPA.

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